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OF THE
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Of The
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and
SEWAGE CONFERENCE

CALGARY, ALBERTA
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WESTERN CANADA POLLUTION CONTROL ASSOC. W.P.C.F.

WESTERN CANADA WATER AND SEWAGE CONFERENCE

PROCEEDINGS

of the

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including the

16TH ANNUAL CONVENTION

of the

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AMERICAN WATER WORKS ASSOCIATION

and the

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of the

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QUALITY ASSURANCE FOR DRINKING WATER PROCESS CONTROL AND ENVIRONMENTAL MONITORING STUDIES

by

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INTRODUCTION

The objective of a laboratory quality assurance (QA) program is to ensure the production of high quality analytical data. A well-designed comprehensive program provides validation that all data are consistently accurate, precise and reproducible. QA programs should be an integral part of overall laboratory operations, particularly in laboratories conducting trace analyses at the ppt or ppb concentration range. When integrated into the laboratory routine, QA results establish laboratory credibility and provide documentation which supports the validity of procedures used, the professional competence of analysts and the reliability of results.

The Glenmore Waterworks Laboratory (GWWL) conducts microbiological, physical and chemical analyses (Table 1) of water samples related to the City of Calgary's potable water. The city operates water treatment plants in two watersheds to serve the population of 640,000. The Glenmore Water Treatment Plant (GWTP), with a design capacity of $5.5 \times 10^5 m^3$/day, obtains its source water from the surface supply Glenmore Reservoir on the Elbow River. The Bearspaw Water Treatment Plant (BWTP), with a design capacity of $7.3 \times 10^5 m^3$/day, obtains source water from the Bearspaw Reservoir on the Bow River. These treatment plants supply a complex distribution system with fourteen secondary reservoirs and 32 pumping stations.

The GWWL staff of eight analyse water samples originating from the watershed, reservoirs, treatment plants and distribution system. Chemical, physical and microbial tests are conducted using standard methodology (1,2) and state-of-the-art analytical procedures developed in the Laboratory. To document the quality of data generated, the GWWL has a comprehensive QA program including sampling protocols, chain of custody procedures, a manual of methods, intra- and interlaboratory testing programs, protocols for corrective action as well as data management and documentation. This paper describes some of the quality control measures applied to sample collection and laboratory analysis of our process control samples and the specialized approach to our source water environmental monitoring studies.

SAMPLING QUALITY ASSURANCE PROTOCOLS

Many factors can affect sample integrity prior to analysis. QA measures ensure that sample collection techniques retrieve a sample representative of the environment desired, that chain of custody procedures, transport and storage are adequate, and that samples are not contaminated, nor are the sample components lost, as a result of these activities. A sampling plan and sample handling protocol must be designed to minimize uncertainties introduced in the sampling process. If these sampling uncertainties are large, results will be unreliable no matter how sophisticated the analytical techniques available to determine sample components. Further, poorly planned sampling sites or sampling frequencies can make the relationship of analytical results to actual conditions impossible to interpret. The sampling aspect of the GWWL QA program is designed to give confidence that samples arrive in the laboratory as similar to their original composition as possible.

TABLE 1

Physical, Inorganic Chemical, Organic Chemical and Microbiological Analysis Capabilities at the City of Calgary Glenmore Waterworks Laboratory

	PARAMETER	METHOD (Reference)	STATUS*
PHYSICAL	Turbidity	Photometer (1)	1
	Color	Nessler Tubes (1)	1
	pH	pH Meter (1)	1
	Conductivity	Conductivity Bridge (1)	1
	Temperature	Mercury Thermometer (1)	1
	Total Dissolved Solids	Calculation (1)	1
	Zeta Potential	Zeta Meter (3)	2,3
INORGANIC CHEMICAL	Alkalinity	Titration (1)	1
	Hardness	Titration (1)	1
	Silica	Spectrophotometry (1)	1
	Anions & Cations	Ion Chromatography (2)	1,2
	Trace Metals	Graphite Furnace Atomic Absorption (4)	
	Mercury	Cold Vapor Atomic Absorption (2)	2,3
ORGANIC CHEMICAL	Trihalomethanes	Headspace, Capillary Gas Chromatography (5)	3
	Volatile Organics	Headspace, Capillary Gas Chromatography (5)	3
	Base-Neutral Compounds	Liquid Extraction Capillary Gas Chromatography (6)	3
	Pesticides and Herbicides	Liquid Extraction Capillary Gas Chromatography (6)	3
	Acids	Liquid Extraction Capillary Gas Chromatography (6)	3
	Phenols	Aqueous Acetylation Capillary Gas Chromatography (7)	2,3
MICROBIOLOGICAL	Heterotrophic Plate Count	Pour Plate (1)	1,2
	Total and Fecal Coliforms	Membrane Filtration (1)	1
	Fecal Streptococci	Membrane Filtration (1)	1
	Sulfate-Reducing Bacteria	Most Probable Number (1)	1
	Legionella	Membrane Filtration (1)	2,3
	Pseudomonas, Staphylococcus	Membrane Filtration (1)	2,3
	Direct Bacterial Counting	Fluorescent Staining (9,10)	2,3
	Giardia	Cartridge Filtration (1)	1
	Chlorophyll 'a'	Fluorometry (11)	1
	Invertebrate Counts	Microscopy	3

* 1. Standard Methods Approved Procedure
2. Published Procedure
3. GWWL Developed Procedure

Treatment Plant and Distribution System Sampling

Errors in the collection of treatment plant process control samples are generally easy to identify. As shown in Figure 1, the City of Calgary GWTP and BWTP contain four and eleven sampling sites within their respective treatment trains.

The in-plant process control sites are monitored at frequencies varying from continuous recording to quarterly analysis. Wide variations in day-to-day results would not be expected according to the 50 years of historical data. Thus within the treatment plants, improperly collected or contaminated samples are easily recognized. Since the laboratory is located within the treatment plant complex, collecting a repeat sample generally poses little problem.

FIGURE 1A

Four Process Control Sampling Sites are Located Within the Glenmore Water Treatment Plant

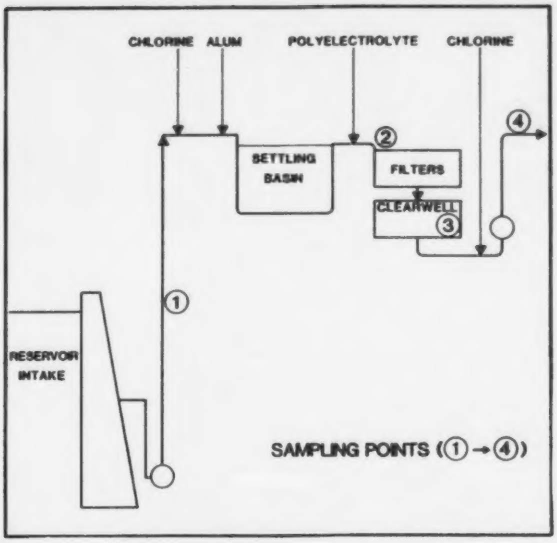
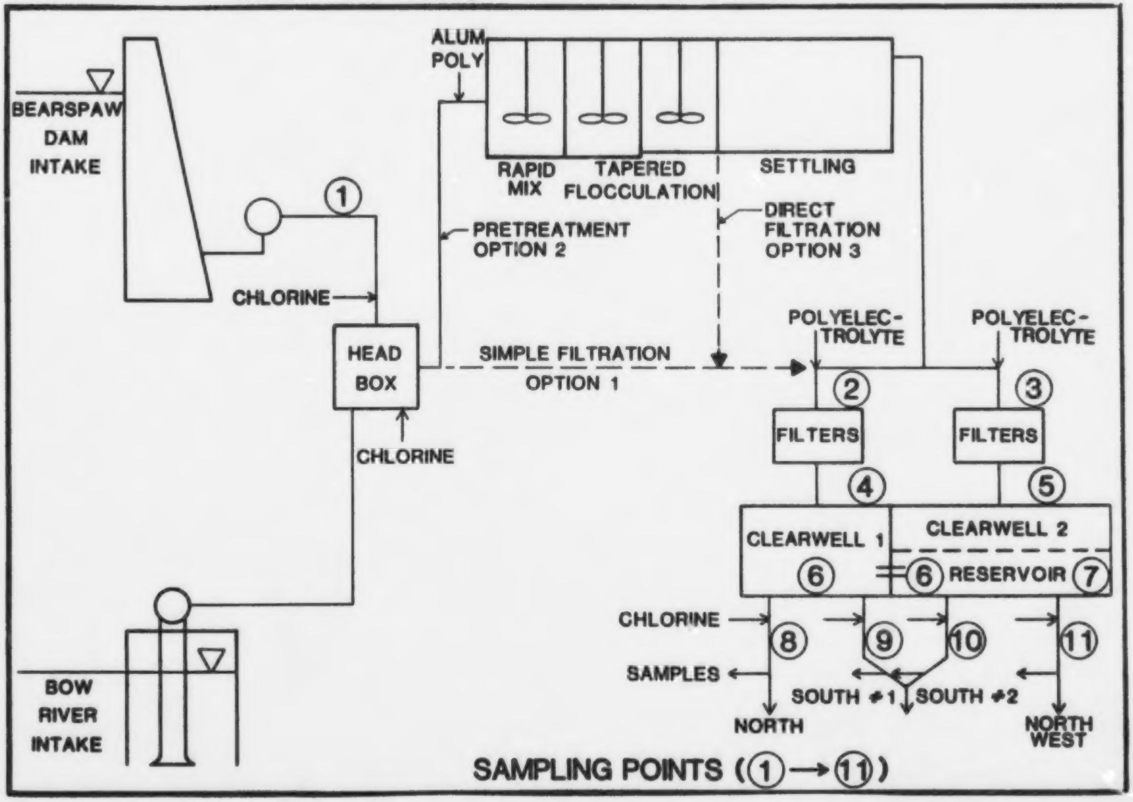

CHLORINE
ALUM
POLYELECTROLYTE
CHLORINE
②
④
SETTLING BASIN
FILTERS
CLEARWELL
③
①
RESERVOIR INTAKE
SAMPLING POINTS (① → ④)

FIGURE 1B

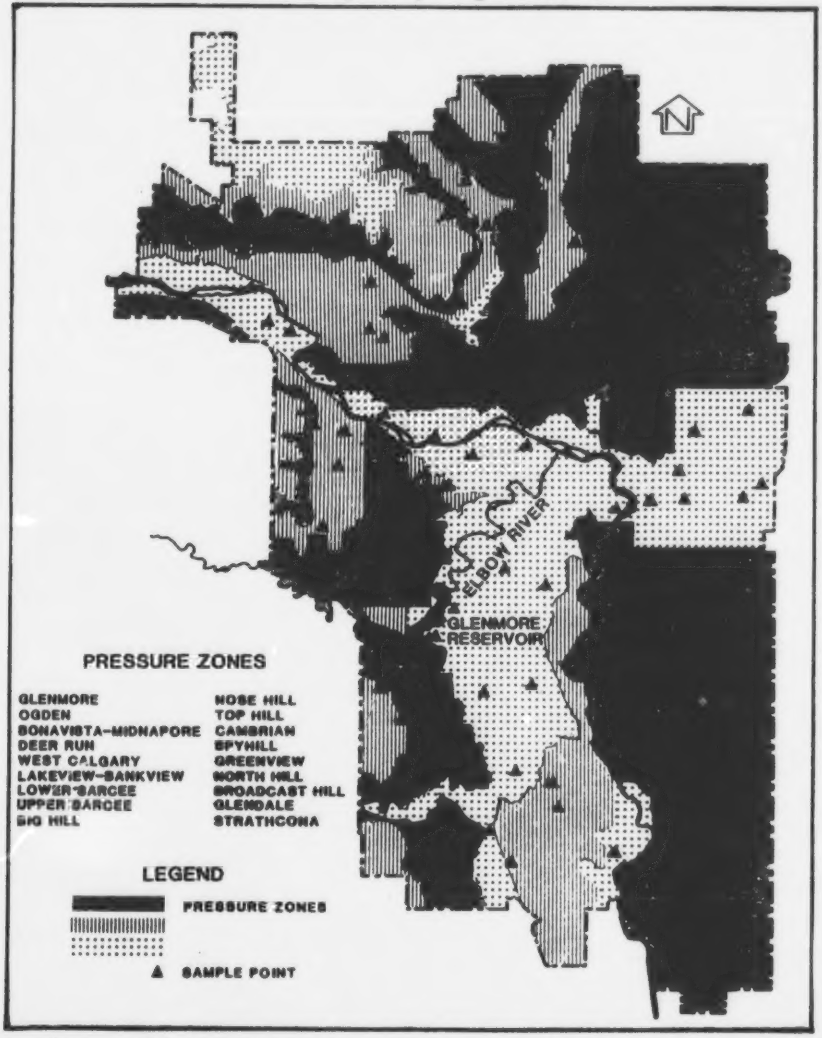
Eleven Process Control Sampling Sites are Located Within the Bearspaw Water Treatment Plant


ALUM
POLY
BEARSPAW DAM INTAKE
RAPID MIX
TAPERED FLOCCULATION
SETTLING
①
PRETREATMENT OPTION 2
DIRECT FILTRATION OPTION 3
CHLORINE
POLYELEC-TROLYTE
POLYELEC-TROLYTE
HEAD BOX
SIMPLE FILTRATION OPTION 1
②
③
CHLORINE
FILTERS
FILTERS
④
⑤
CLEARWELL 1
CLEARWELL 2
⑥
⑥ RESERVOIR ⑦
CHLORINE
⑧
⑨
⑩
⑪
SAMPLES
SOUTH #1
SOUTH #2
NORTH
NORTH WEST
BOW RIVER INTAKE
SAMPLING POINTS (① → ⑪)

Errors in the collection of distribution system samples are not as easy to recognize as are treatment plant samples; distribution system sites are also more difficult to resample. The GWWL has a network of sampling sites throughout the city selected to pinpoint any deterioration in water quality after treatment. The locations, plotted in Figure 2, include all pressure zones. The number of sites per pressure zone is proportional to the number of residents in the zone. The appropriateness of each site location is reviewed annually.

FIGURE 2

City of Calgary Distribution System
Quality Sampling Network


ELBOW RIVER
GLENMORE RESERVOIR
N
PRESSURE ZONES
GLENMORE
OGDEN
BONAVISTA-MIDNAPORE
DEER RUN
WEST CALGARY
LAKEVIEW-BANKVIEW
LOWER SARCEE
UPPER SARCEE
BIG HILL
NOSE HILL
TOP HILL
CAMBRIAN
SPYHILL
GREENVIEW
NORTH HILL
BROADCAST HILL
GLENDALE
STRATHCONA
LEGEND
PRESSURE ZONES
SAMPLE POINT

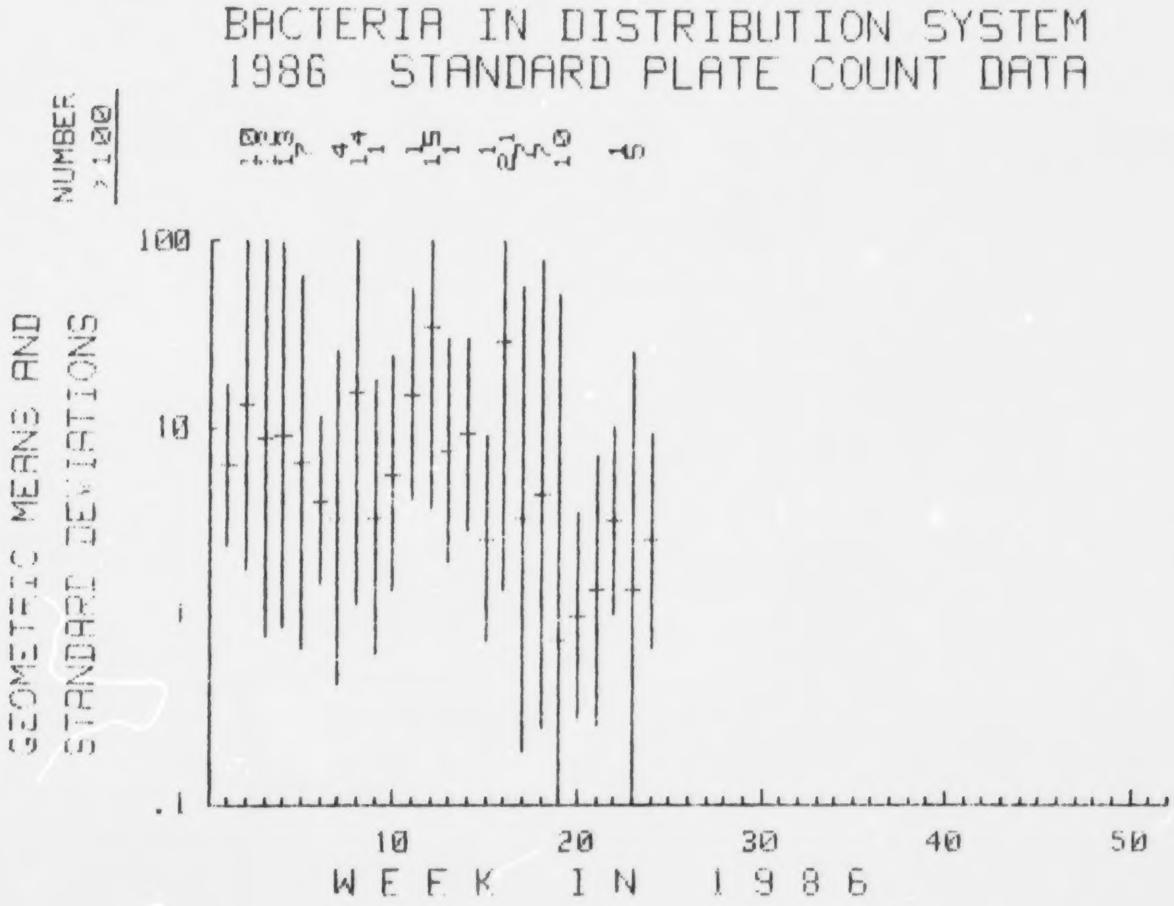
From these sites, 62 samples per week are collected and submitted to the Provincial Laboratory of Public Health (Calgary) for enumeration of total coliforms and heterotrophic plate count. Fifty of the 62 samples are collected from fixed locations; the remaining 12 are collected from 24 alternating sites. Ten of the fixed locations serve as the sites for weekly chlorine tests, biweekly trihalomethane monitoring and monthly chemical monitoring (pH, alkalinity, hardness, conductivity, turbidity and temperature).

Quality control measures are applied to each of these tests. At least 10% of samples are collected and analysed in duplicate; standard deviations outside a predetermined range warrant investigation. All samples for laboratory analysis are stored in an iced cooler and are analysed within four hours of collection. Instruments used for field analysis are subjected to daily calibration checks and weekly or monthly recalibration.

Data gathered in heterotrophic plate count enumerations are grouped by week and compared to previous data by one way analysis of variance on the log-transformed counts. A plot of these results is shown in Figure 3. A quick visual inspection of the graph allows the analyst to determine if the data are significantly ($p < 0.05$) or not significantly different ($p > 0.05$) from previous data. A significant difference would warrant prompt investigation of treatment plant disinfection efficacy.

FIGURE 3

Computer Printout of Weekly Distribution System Standard Plate Count (cfu/mL) Data Shows Geometric Means and Standard Deviations. The Number of Sites with Counts Greater than 100 cfu/mL are Noted on the Graph.



Environmental Monitoring Sampling

In any attempt to assess environmental water quality, objectives must be clearly defined. The number and location of sampling sites and sampling frequency must be carefully formulated to achieve the desired goals. Many environmental factors such as temperature, wind, sun, rain and flow, affect a surface water body; great natural variability can occur in chemical, physical and microbiological analytes daily. Thus, "outliers" or contaminants are not easily differentiated from real sample variability. Special efforts must be made to demonstrate that each sample collected is representative of the area from which it was taken.

Three recent water quality monitoring studies with various objectives have been conducted in the Glenmore and Bearspaw Reservoirs (12-14). To meet the objectives of each study, different sampling strategies were used as summarized in Table 2. From the results of preliminary reconnaissance samplings to evaluate spatial and temporal variability, permanent networks of fixed sampling stations were established in the Glenmore (Figure 4) and Bearspaw (Figure 5) Reservoirs and their respective source rivers, the Elbow and the Bow Rivers. Point source sampling was used to monitor storm sewer input in the Glenmore Reservoir while the fixed station network provided an intensive water characterization survey. Grab samples rather than composite samples were analyzed from all sampling sites and depths.

TABLE 2

Sampling Strategies for Environmental Water Monitoring Studies in the Glenmore and Bearspaw Reservoirs

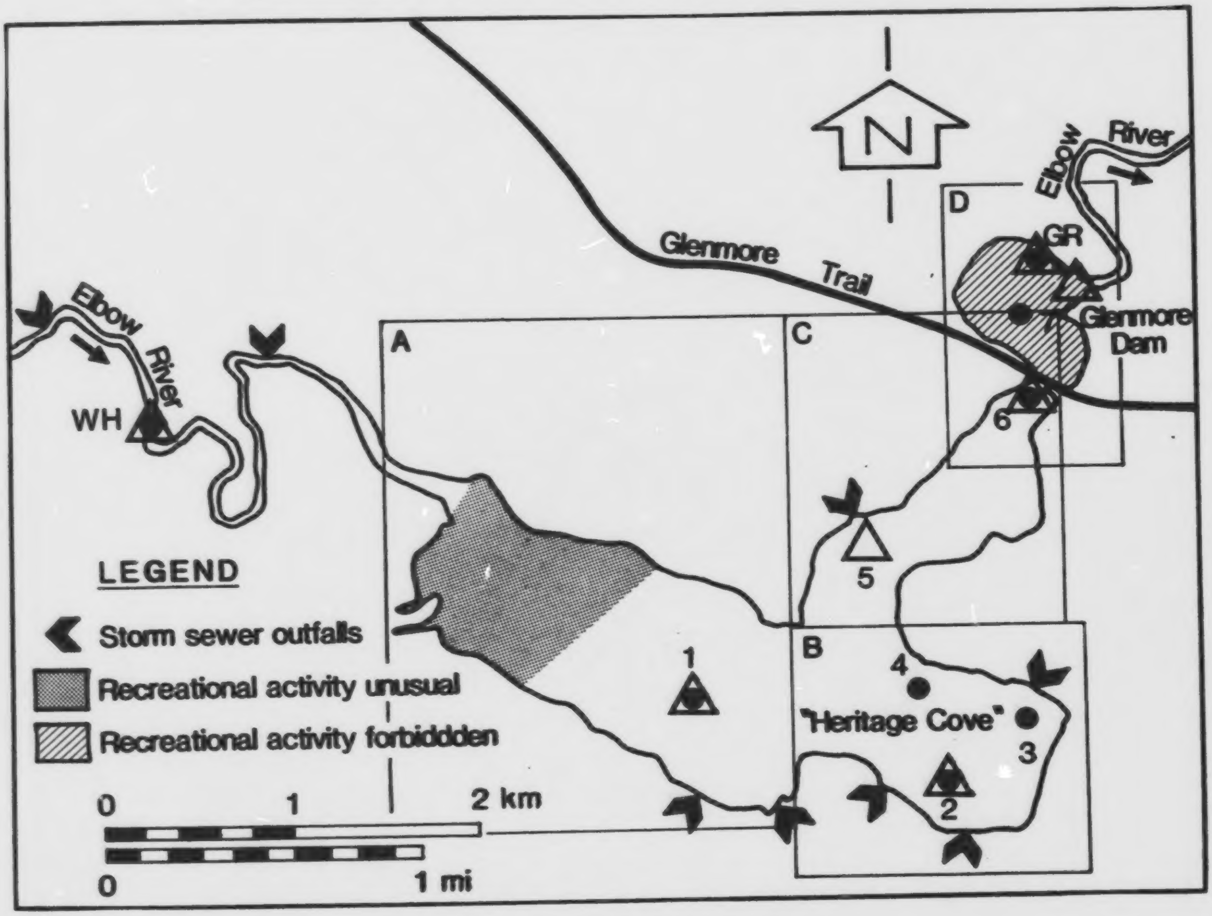
Sampling Strategy	Sampling Frequency	Purpose
Reconnaissance	Random	Preliminary sampling to identify key reservoir sampling points for reservoir monitoring.
Point Source Characterization	Before, during and after rainfall.	Storm event and urban runoff monitoring.
Network of Fixed Stations	Intensive monitoring survey.	Reservoir water quality monitoring.

Field blanks are handled along with each sample set, the sample set being defined as samples collected from the same general area at approximately the same time. The field blank is prepared by bringing a carboy of laboratory pure water (MilliQ, Millipore Ltd., Bedford MA) into the field. The laboratory pure water is poured through the water sampling device (e.g., a Kemmerer, Nisken or Nansen sampler) and dispensed into sample bottles in the same manner as actual water samples. The field blank samples remain in the field under the same conditions as the bona fide samples. Wherever a set of samples is shipped or stored, it is accompanied by appropriate field blanks. Thus, any contamination occurring in the field, in storage, or in transport can be discovered. A field blank can often help to distinguish trace sample artifacts from existing ppt analyte levels. During the Glenmore and Bearspaw

Reservoir studies, for example, samples for trace metal analyses were stored in nitric acid-washed polyethylene vials. Even though the vials were rigorously cleaned, the field blank revealed that ppb levels of zinc were leaching from the containers into the samples. If only a laboratory blank had been prepared to calibrate the atomic absorption spectrophotometer, this zinc sample contamination might have been misinterpreted as environmentally significant.

FIGURE 4

Glenmore Reservoir and Elbow River
Environmental Monitoring Sampling Site Locations

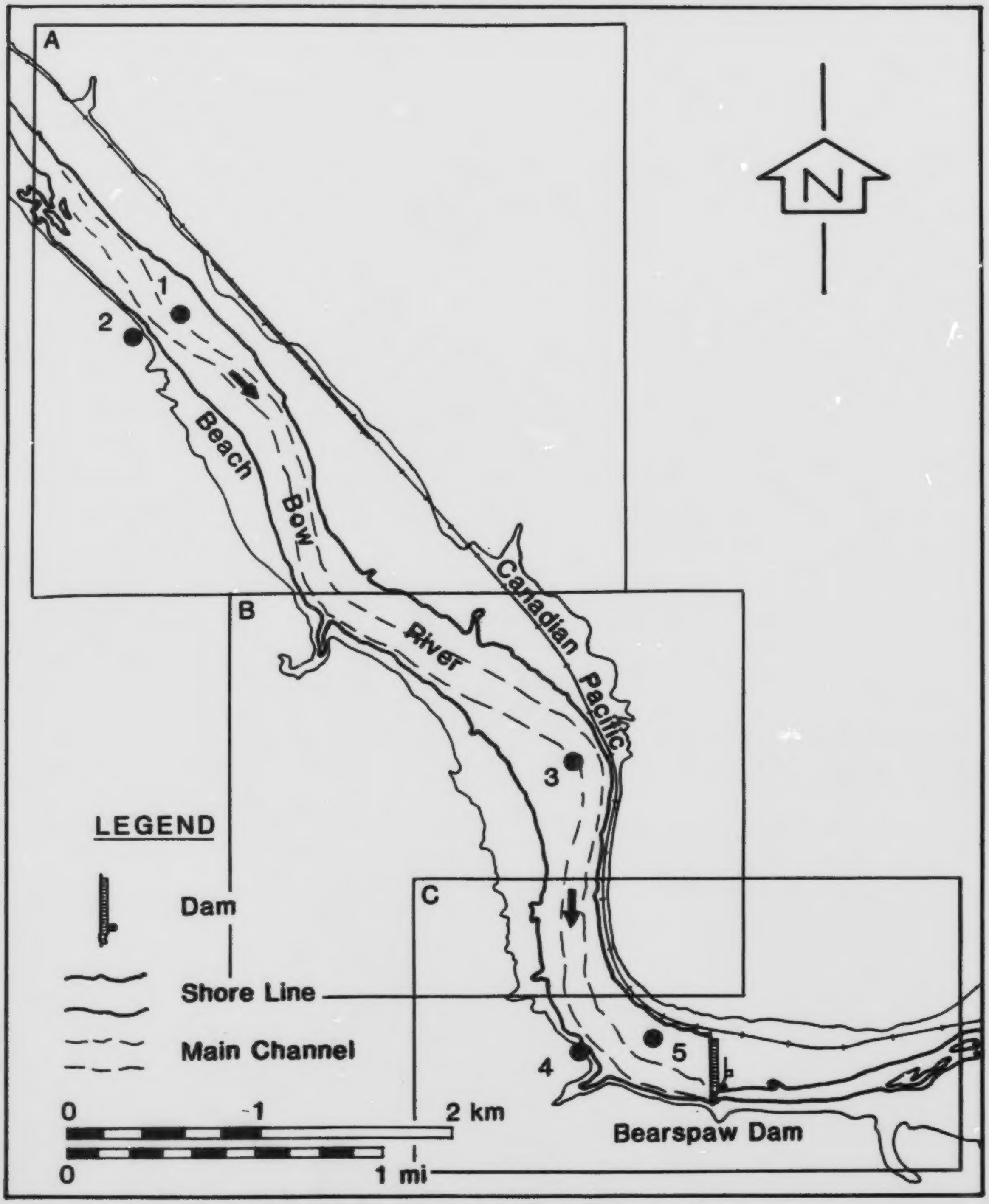

N
Elbow River
Glenmore Trail
D
GR
Glenmore Dam
A
C
Elbow River
WH
6
5
LEGEND
Storm sewer outfalls
Recreational activity unusual
Recreational activity forbiddden
B
4
1
"Heritage Cove"
3
2
0
1
2 km
0
1 mi

● 1984 GLENMORE RESERVOIR SAMPLING SITES

△ 1985-86 GLENMORE RESERVOIR SAMPLING SITES

FIGURE 5

Bearspaw Reservoir and Bow River
Environmental Monitoring Sampling Site Locations


A
B
C
N
1
2
3
4
5
Beach
Bow
River
Canadian Pacific
LEGEND
Dam
Shore Line
Main Channel
0
1
2 km
0
1 mi
Bearspaw Dam

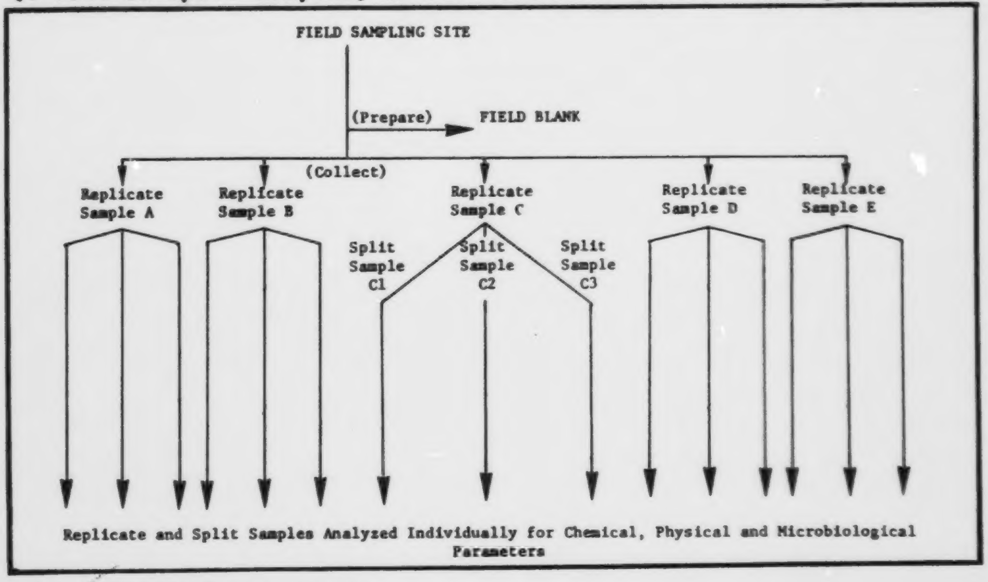
● SAMPLING SITES (1985-86)

BEARSPAW RESERVOIR, CALGARY

The environmental water monitoring studies conducted by the GWWL necessitated the development of specialized sampling control checks. To demonstrate sampling reproducibility, five replicate samples were collected sequentially at one sampling site. Each replicate was further split into three subsamples as shown in Figure 6. A field blank was also collected. These 16 samples were then analyzed for all physical, chemical and microbiological parameters routinely tested for in reservoir samples. Analytical variability was assessed using a t-test on the data from the three split samples. Sampling variability was assessed using a one-way analysis of variance on data from the five replicate samples. Any contamination of samples was determined from the field blank readings.

FIGURE 6

QA Design to Establish Sampling Reproducibility (ANOVA on Replicate Samples) and Analytical Variability (t-test on Split Samples) in Environmental Water Quality Studies


FIELD SAMPLING SITE
(Prepare)
FIELD BLANK
(Collect)
Replicate Sample A
Replicate Sample B
Replicate Sample C
Replicate Sample D
Replicate Sample E
Split Sample C1
Split Sample C2
Split Sample C3
Replicate and Split Samples Analyzed Individually for Chemical, Physical and Microbiological Parameters

Sample Storage and Preservation

Water samples collected from any source are unstable. Even using accepted sample preservation practices such as those established by the U.S. Environmental Protection Agency (2), complete stability of constituents can never be achieved. The interval between sample collection and analysis may introduce changes in the concentration or physical state of the constituents to be measured.

The applicability of preservation procedures to various water samples must be documented for each parameter and each sample type. For example, samples for silica, hardness and conductivity measurements can reportedly (2) be stored for up to 28 days, 6 months, and 28 days, respectively, without significant deterioration in measured values. These are relatively long shelf lives. However, we observed a significant difference ($p < 0.05$) in measured values after a 24 hour storage, as shown in Table 3. Paired t-tests at the 0.05 level of significance indicated significant differences in all three analyses after 24 hour storage. Appropriate QA samples were analyzed with both runs to eliminate analyst and instrument variability as the source of the measurement differences. Thus, expedience in sample throughput is important even for analytes with long reported storage stabilities. If tests cannot be conducted immediately, storage times for various analyses should be comparable for all samples to minimize storage artifacts. There are no acceptable preservation methods or storage time for microbiological parameters.

TABLE 3

Measured Values for Hardness, Conductivity, and Silica in Replicate Samples Immediately After Sample Collection (t = 0) and After Storage for 24 Hours Under Recommended (2) Conditions (t = 24h)

SAMPLE NO.	HARDNESS (mg/L as Ca CO_3)		CONDUCTIVITY (us/cm)		SILICA (mg/L)	
	t = 0	t = 24h	t = 0	t = 24h	t = 0	t = 24h
1	174.8	175.0	348	342	4.22	3.97
2	173.4	174.8	348	343	4.02	3.97
3	174.4	175.8	349	343	4.12	3.92
4	172.8	175.0	348	343	4.27	3.97
5	175.6	176.4	348	342	--	--
6	175.0	174.4	348	343	--	--
7	174.2	176.2	348	343	--	--
Mean	174.3	175.4	348.1	342.7	4.16	3.96
Variance	.92	.58	.14	.24	.01	.01
Standard Deviation	.96	.76	.38	.49	.11	.03
95% Confidence Interval						
Lower Limit	173.4	174.7	347.8	342.3	3.98	3.92
Upper Limit	175.2	176.1	348.5	343.2	4.33	4.00
Paired t-test at $\alpha = 0.05$	Significant Difference		Significant Difference		Significant Difference	

ANALYTICAL QUALITY ASSURANCE PROTOCOLS

A written manual and record system is of particular importance to analytical quality assurance. The manual should specify methods and procedures currently in use. The associated record system includes instrument repair and maintenance logs, performance specifications for instrument consumable parts, detection limits, precision expectations and method reagent preparation guidelines. A QA manual is absolutely necessary to document and justify the use of nonstandard or newly developed procedures. Permanent records of QA test results should be archived along with accumulating analytical data.

Chemical Analyses

Variability is associated with all measurements among analysts in the same laboratory and even greater variability exists where ppt level analyses are conducted over the span of several years by a transient group of analysts. Quality control samples add an increased analytical workload to any laboratory which already processes a large number of samples and a broad range of tests. Thus, each additional QA sample introduced must be cost effective and add a clear benefit to the laboratory QA program. Since the total number of determinations which can be accommodated by laboratory budget and staff time are limited, the inclusion of inappropriate quality control samples could actually reduce effective laboratory analysis time. It is important to employ quality control measures which will add confidence rather than detract from the validity of the accumulating data base.

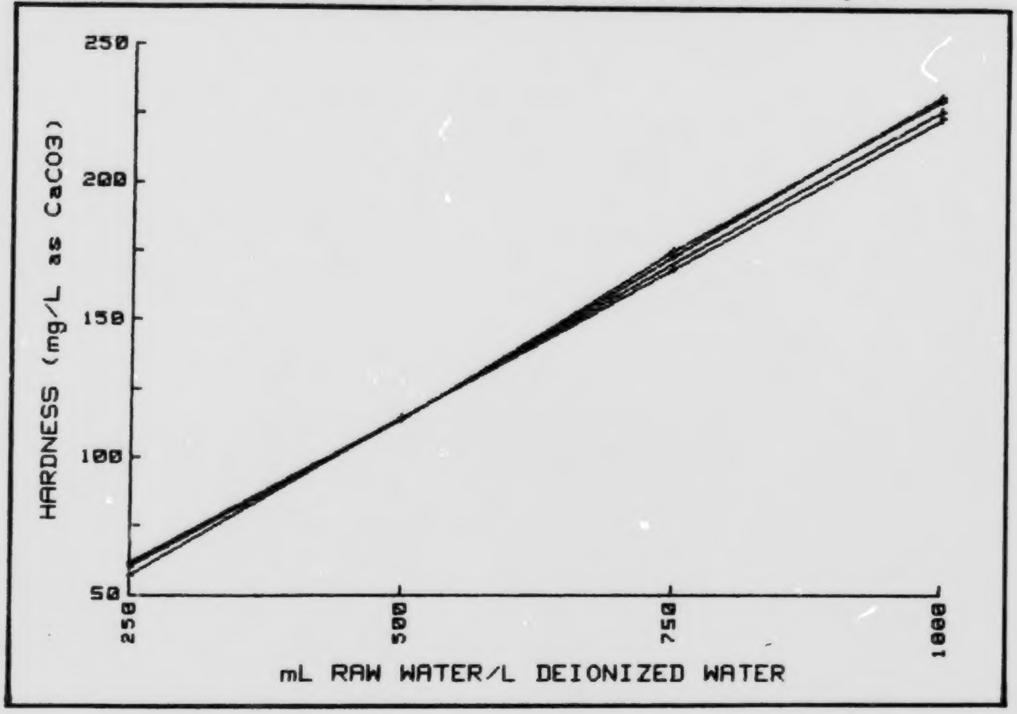
At the GWWL we incorporate several types of quality control samples into the chemical and physical analyses listed in Table 1. Reagent blanks are prepared prior to actual analyses to calibrate instrumentation and correct for background response to reagents. The inclusion of the field blank previously discussed is an essential part of sample collection quality control. Results of duplicate analyses provide documentation of the reproducibility of results. At least one duplicate is run every ten samples to verify precision of the method. Acceptable ranges for the standard deviation of results of duplicate analyses are assigned on the basis of known method precision as previously established in the laboratory. Standard deviations are calculated and documented for all currently used methods.

A very effective and inexpensive method of preparing quality control samples is by serial dilution of actual samples. Figure 7 shows the graph obtained by serial dilution analyses for hardness. A Glenmore Reservoir sample was serially diluted with deionized water to prepare a series of five samples. Each dilution was then divided into three replicate samples, all of which were slipped into the analysis routine. The resulting graph gives visual confirmation that the methodology and analysts are capable of determining levels of hardness only 25% of the expected levels. The mathematical operations performed on the replicates (Figure 7), demonstrate that precision (reproducibility) decreases as the concentration of analyte of interest decreases but that the accuracy (difference between true and calculated values) remains acceptable. Serial dilution can only be applied to analytes present at concentrations well above method detection limits. The advantage of such samples is that, firstly, analytes are measured in a matrix composed in part of actual sample water. Any interferences within the sample itself which cause analyte suppression or cross reaction will produce a non-linear dilution graph. Secondly, the method reliability is effectively checked over a range of concentrations.

For analyses of parameters present at near the reliable detection limit or to identify loss of sample constituents, replicate, duplicate, split, or serial dilution samples are not effective quality assurance strategies. At the GWWL, QA samples prepared by adding known analyte concentrations to water are used for this purpose. For trace metal analyses, ions and all trace organic compound analyses, "spiked" samples are run as independent checks of technique, methodology and standards. One natural sample is collected in duplicate for each analysis to be performed. To one of the duplicates a spike of known value is added. Both samples are stored and analyzed together; the percent recovery of the spike is assessed. A marked decrease of the spike concentration would lead to a thorough investigation to determine the cause of loss. All samples collected with the deteriorated batch of spiked samples would be discarded and recollected for analysis.

FIGURE 7

Serial Dilution Quality Assurance for Hardness Analyses



SAMPLE		HARDNESS RESULTS		
Dilution (mL Raw Water per L Deionized Water)	Replicate	Calculated Value	Recovered Value	Calculated: Recovered[1]
	A	-	229.2	-
0	B	-	230.6	-
	C	-	225.2	-
	D	-	222.2	-
	A	171.9	174.2	0.98
750	B	173.0	172.4	1.00
	C	168.9	-	
	D	166.7	167.8	0.99
	A	114.6	114.0	1.00
500	B	115.3	113.6	1.01
	C	112.6	114.5	0.98
	D	111.1	114.4	0.97
	A	57.3	60.6	0.95
250	B	57.7	60.0	0.96
	C	56.3	61.4	0.92
	D	55.6	56.8	0.98

---

[1] Ratio of Calculated Value to Recovered Value

In addition, samples of known concentrations prepared by independent laboratories are used to validate technique, methodology and calibrations. The GWWL makes use of the QA sample program provided by the U.S. Environmental Monitoring and Support Laboratory (15). If results for quality control check samples fall outside the established 95% confidence interval for the analyte, results for all samples concurrently analyzed are discarded and analyses repeated. In addition, the GWWL participates in Round Robin studies for parameters of interest under the auspices of the Alberta Water Analysts Committee.

At the GWWL, part of the QA evaluation is directly incorporated into the calculation of results whenever possible. The ion chromatographic (IC) analysis of samples yields a good example of this. A computer program has been devised to perform this function. The analyst enters into the program the data from the IC analysis and accompanying pH, conductivity and temperature tests. The program calculates the ion balance based on ion concentration, as well as based on EDTA hardness, and the ratio of TDS:specific conductance. The anion and cation sums should be balanced, the measured specific conductance should be approximately 100 times the anion or cation sum when expressed as meq/L, and the calculated total dissolved solids should be a known percentage of the specific conductance. The computer printout (Figure 8) lists the acceptable ranges for each of these values. Falling outside the range on any variable necessitates the rerunning of the samples in question.

FIGURE 8

Computer Printout of Calculations
Performed on Ion Chromatographic Data for
Quality Assurance Purposes

```
MEASURED VARIABLES
pH =               8.27          Specific Conductance =    317
Temperature =      15.2°C        EDTA hardness =           165.4 ppm CaCO3
Ca =               48.9 ppm      P. Alk =                  0 ppm
Mg =               12.1 ppm      T.Alk =                   133 ppm
Na =               1.27 ppm      SO4 =                     32.5 ppm
K =                0.56 ppm      Cl =                      1.15 ppm
NH4 =              0.02 ppm      F =                       0.27 ppm
                                 NO3 =                     0.16

CALCULATED VARIABLES
Hardness =  171.9 ppm CaCO3
CO3 =       0 ppm
HCO3 =      162.1 ppm
TDS =       180.7 ppm

QUALITY ASSURANCE CALCULATIONS
Based on ions:                      Based on EDTA:
   Σ cations =       3.51 meq             Σ cations =         3.38 meq
   Σ anions =        3.38 meq           * Ion balance =       1 *
   * Ion balance =  1.04 *                EDTA/Calc ratio =  0.96
  ** TDS/sc ratio=  0.57 **
                                 ACCEPTABLE RANGE FOR *  0.90 to 1.10
                                 ACCEPTABLE RANGE FOR ** 0.55 to 0.70
```

Specific Microbiological Testing Strategies

Unlike chemical analysis, microbiological analyses measure living organisms which continually change in response to their environment. Analytical standards, prepared samples of known value and spiked samples are unavailable for the microbiological parameters listed in Table 1. Thus, equipment performance monitoring (1,16,17) (atmosphere, temperature, sterilization effectiveness, timing accuracy) is of the utmost importance in QA for microbiological testing. In addition to standard equipment monitoring, the GWWL has implemented a rigorous program to control analytical procedures and analyst variability.

The performance of culture media is critical to all microbiological analyses. At GWWL, commercially-prepared medium is used whenever possible. Manufacturer's expiration dates are strictly followed. Each bottle of medium is dated upon receipt and the lot number recorded. The bottle is stored in proper conditions, and is dated upon opening. Details of the preparation of every batch of medium (e.g. date prepared and used, pH, preparer's name) are recorded in a permanent logbook.

Before a new lot of culture medium is put into service, its recovery of microorganisms is compared to the old lot using samples of source and finished waters. A minimum of five replicates are plated onto the old (reference) and new (test) media. Colony size and appearance ARE visually compared and the number of colonies recovered is enumerated. The number of colonies enumerated from both media is statistically evaluated using the Students t'test on the log-transformed counts. If the media produce counts that are not significantly different ($p > 0.05$), the new lot is put into service. If the media do have significantly different recoveries ($p < 0.05$), we refer to the manufacturer's recommendation for further tests. These include a visual comparison of the two dehydrated powders, testing the pH of a known weight of medium dissolved in a known volume of solute and characteristics of type culture strain organisms plated on the two lots.

QA for microbiological analyses includes blanks and duplicates, positive and negative controls, and verifications of colonies yielding positive and negative results. Blanks are run at the beginning and end of each membrane filtration run and on each batch of SPC medium. Duplicates are run on 10% of the samples tested for each parameter. These duplicates are recorded in the QA book and are periodically evaluated using statistical tests. The appearance and recovery of type culture strains are periodically tested. Colonies recovered in membrane filtration tests are verified as positive, negative, false positive, or false negative using simple biochemical tests (1).

The microbiological QA program includes checks to minimize between-analyst variability. In spot checks, technicians recount plates counted by other technicians. Any discrepancies are pointed out, and the spot check frequency is intensified until the counts of both analysts are within 10% of each other. In addition, twice annually (and more often if substantial variability is observed) a formal QA test is run. A set of plates for each analysis run are prepared and each analyst counts all plates. Paired t-tests are applied to the logs of the counts to establish the level of significance of the counts.

The microbiological analyses are also subjected to an interlaboratory QA program. Large volume samples are collected and split into two replicate samples. One replicate is run in the GWWL, the other by the Provincial Laboratory of Public Health (Calgary). Results are compared and statistically evaluated. If a large difference in results for any test is observed, culture medium used by the two laboratories is exchanged. Replicate samples are retested by each laboratory on both lots of media. Additional QA measures include verification of colonies by microbiologists at both labs and tests on type strains of organisms.

CONCLUSION

Quality assurance starts with sample collection and does not end until results are reported. Each field and laboratory activity which has any relation to the sampling strategy and sample handling must be proven not to affect the sample or bias results. The analytical facet of the QA program must provide documentation which supports the validity of procedures used, the professional competence of analysts, and the reliability of results. All quality control measures must be clearly defined in an in-house quality assurance manual which outlines all aspects of sample analysis. Without that documentation, the laboratory's credibility is seriously at risk. QA programs for no two laboratories will be identical but all must have a firm basis and provide legally defensible validation that data is accurate, precise and reproducible.

REFERENCES

1. American Public Health Association. 1985. Standard Methods for the Examination of Water and Wastewater, 16th ed. American Public Health Association, Inc., Washington, D.C.

2. United States Environmental Protection Agency. 1983. Methods for Chemical Analysis of Water and Wastes. Environmental Monitoring and Support Laboratory, Cincinnati, Ohio. EPA-600/4-79-020.

3. Letterman, R.D., S.G., Vanderbrook and P. Sricharoenchaikit. 1982. Electrophoretic mobility measurements in coagulation with aluminum salts. J. A. Waterworks Assn. Jan. 1982:44-51.

4. Routh, M.W., P.S. Doidge, J. Chidzey and B. Frary. 1982. Advances in graphite furnace atomization. Am. Lab., May 1982.

5. Hargesheimer, E.E., and S.L. Milkert. 1986. Computer automated headspace analysis as an alternative to purge and trap for volatile organic analysis. In preparation.

6. Hargesheimer, E.E. G.A. Irvine, A. Badakhshan and R.T. Seidner. 1986. Pilot studies into effects of disinfection strategies on drinking water quality. Proc. 2nd Nat. Conf. Drinking Water. Edmonton, Alberta.

7. Coutts, R.T., E.E. Hargesheimer and F.M Pasutto. 1979. Gas chromatographic analysis of trace phenols by direct acetylation in aqueous solution. J. Chromat. 179:291-299.

8. Brodsky, M.H. and B.W. Ciebin. 1978. Improved medium for recovery and enumeration of *Pseudomonas aeruginosa* from water using membrane filters. *Appl. environ. Micro*. 36:36-42.

9. Zimmerman, R. and L.A. Meyer-Reil. 1974. A new method for fluorescent staining of bacterial populations on membrane filters. *Kiel Meeresforsch.* 30:24-27.

10. Porter, K.G. and Y.S. Feig. 1980. The use of DAPI for identifying and counting aquatic microflora. *Limnol. Oceanogr*. 25:943-948.

11. Yentsch, C.S. and D.W. Menzel. 1963. A method for the determination of phytoplankton, chlorophyll and phaeophytin by fluorescence. *Deep-Sea Res*. 10:221.

12. Marshall, Macklin, Monaghan Limited. 1985. *Elbow River Watershed Study*. Report to the City of Calgary, Engineering Department.

13. Hargesheimer, E.E. and C.M. Lewis. 1985. *1984 Glenmore Reservoir Summer Water Quality Study*. Internal report. City of Calgary, Engineering Department.

14. Hargesheimer, E.E. and C.M. Lewis. 1986. *Comparative Study of Glenmore and Bearspaw Reservoirs*. Internal report (in preparation). City of Calgary, Engineering Department.

15. Quality Control Samples, United States Environmental Protection Agency, Environmental Monitoring and Support Laboratory, Quality Assurance Branch, Cincinnati, Ohio. 45268, U.S.A.

16. Geldreich, E.E. 1975. *Handbook for Evaluating Water Bacteriological Laboratories*, 2nd ed. United States Environmental Protection Agency, Cincinnati, Ohio.

17. Association of Official Analytical Chemists. 1986. Short course: Quality assurance for microbiological laboratories. Seattle, Washington.

CROSS CONNECTION CONTROL
FIRE PROTECTION SYSTEMS - CROSS CONNECTIONS THAT REQUIRE BACKFLOW PROTECTION

by

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Febco Backflow Prevention Devices Fresno, CA

Fire Protection Systems - A cross connection requiring backflow prevention.

Water purveyors and health officials today commonly protect the potable water supply from backflow due to back siphonage and/or back pressure.

All pressurized or "water upon demand" systems can cause backflow contaminates to enter the supply system when the supply pressure becomes lower than the system pressure and is not protected by backflow prevention.

An automatic fire sprinkler system is a "water upon demand system" intended to control and extinguish fires. The fire sprinkler heads are temperature sensitive and open when exposed to heat. This opening allows water to flow out of the sprinkler head and extinguish the fire. The water utilized is most commonly carried from the public and/or auxiliary supply by uncoated black iron pipe.

The basic components of a fire sprinkler system are: 1] Public and/or an auxiliary supply main. 2] Fire service into the building. 3] A Check valve at the point of connection. 4] The fire system piping to the sprinkler heads and/or standpipe. 5] A fire department pumper connection to allow additional pressure to "push" the water faster to the fire.

The common areas of concern to water purveyors and health officials about backflow of fire sprinkler water are:

1. Black water caused by the leaching of heavy metals such as zinc, cadmium, iron or lead into water which stands in pipelines for long periods.
2. Rust and scale.
3. Wetting agents and chemicals [rust inhibition]
4. High hazard antifreeze [ethylene glycol]
5. Foam systems.
6. Connection to polluted or contaminated secondary sources.
7. Unauthorized use of water [tapping of fire mains for unknown purposes, which may be cross-connected to various hazards.

The American Water Works Association has classified fire protection systems on the basis of water source and arrangement of supplies in order to help determine if backflow prevention devices are required and if so, which type.

Class 1 - Direct connection from public water mains only; no pumps, tanks or reservoirs; no physical connection from other water supplies; no antifreeze or other additives of any kind; all sprinkler drains discharging to atmosphere, dry wells or other safe outlets.

Class 2 - Same as Class 1, except that booster pumps may be installed in the connections from the public supply main.

Class 3 - Direct connection from public water supply main, plus one or more of the following: elevated storage tanks; fire pumps taking suction from above ground covered reservoirs or tanks; and pressure tanks (all storage facilities are filled or connected to public water only, the water in the tanks to be maintained in a potable condition. (Otherwise, Class 3 systems are the same as Class 1).

Class 4 - Directly connected from public mains similar to Class 1 and 2, and with an auxiliary water supply on or available to the premises; or an auxiliary supply may be located within 1,700 feet of the pumper connection.

Class 5 - Directly supplied from public mains and interconnected with auxiliary supplies, such as: pumps taking suction from reservoirs exposed to contamination or rivers and ponds; driven wells; mills or other industrial water systems; or where antifreeze or other additives are used.

Class 6 - Combined industrial and fire protection systems supplied from the public water mains only, with or without gravity storage or pump suction tanks.

Until recently, Class 1 and Class 2 systems have not generally required backflow protection at the service connection. There are many water purveyors and health officials, however, that have required backflow preventers under the following conditions:

1. The sewer and fire lines are in a common trench and/or the separation distances are not met.

2. A looped fire system is providing water from more than one service.

3. When the fire department is unwilling to state that it will not use chemical additives.

4. When auxiliary water is within 1,700 feet or is designated as secondary supply regardless of its distance.

5. Any fire system in a building where the domestic system is protected and there may be potential cross-connection.

6. Any fire system in a structure where toxic or hazardous chemicals are processed.

7. Any fire sprinkler system in a high-rise building. (Building 75 ft. high measured from the floor of fire department access to the highest floor of human occupancy.)

8. When the fire department connection is on the water supply side of the check valve.
9. When there is a change of occupancy in the building.

10. In large, complex systems where it is difficult to verify that a cross-connection does not exist.

It is interesting to note that recently one large and populous county in Southern California has determined on advice by its legal counsel that any automatic fire system with a fire department connection is nether Class 1 nor Class 2, but rather should be classified a Class 4 or Class 5 system.

Determining the classification of a fire system can be difficult and the exceptions to the Class 1 and Class 2 systems when backflow prevention might be allowed not only can create confusion, but does not answer what type of backflow prevention device should be utilized in each classification. When a regulatory agency establishes a cross connection control ordinance, it is important to establish protection requirements which are commensurate with the degree of hazard. The same standards should be required on all "water upon demand" systems whether the system is plumbing, irrigation or fire sprinkler. The Pacific Northwest Section - AWWA has a cross connection control committee which has established a guideline for minimum backflow protection recommendations for the different types of automatic fire systems, according to the degree of hazard as well as water source and piping arrangement. This guide is a practical and thorough procedure for backflow protection requirements for fire protection systems.

1. Low Hazard Fire Systems

   A single non-pumped fire system serving a building 30 feet in height or less. With or without a fire department pumper connection which is at least 1,700 feet from an auxiliary water supply.

   Minimum Backflow Protection Required

   > Single detector check meter, alarm check valve or single soft seated check valve (non-approved backflow preventers).

   With the introduction of the spring loaded, centre stem guided detector check valve, there is now a better seating check valve to pump additional pressure against. While this check provides tighter seating than older style check valves, they are not approved as backflow preventers so should only be utilized in extremely low hazard fire systems.

2. Moderate Hazard Fire Systems

   A. Wet or dry system with pumper connection and/or within 1,700 feet of an auxiliary water supply.

   B. Wet systems with an in-line booster pump or buildings where water piping exceed 30 feet above finished grade.

   C. Any system with private hydrants.

   D. Any looped system (an intertied system with more than one service connection).

   E. Systems with pumper connections where corrosion inhibitors or other chemicals are added to tanks of fire trucks.

Minimum Backflow Protection Required

Double check valve assembly or detector double check valve assembly.

Approved backflow prevention assemblies are required to pass very stringent requirements for that approval, especially for usage in fireline applications. The typical approval agency requirements should include U.S.C., CSA, ASSE, as well as U/L and F.M. The double check valve assembly has proven to be reliable protection against backflow in moderate hazard applications. The spring loaded center stem guided checks can be installed vertically or horizontally for ease of installation. These assemblies offer very low pressure drop characteristics at low flow requirements for the vast majority of fires as well as at high flow requirements for total system calculations. The detector double check offers not only approved backflow protection, but because of heavier spring loading, forces all low flows (up to 8-10 gpm) through a protected bypass assembly to meter any unauthorized water usage or system leaks. Any flows beyond the low range will open the main line assembly to allow for any fire flow requirements.

3. High Hazard Fire Systems

   A. All foamite installations.

   B. Systems where an unapproved water supply is permanently connected to a fire system.

   C. Systems in which antifreeze is allowed.

Minimum Backflow Protection Required

Reduced pressure principle backflow preventer assembly.

"RP" assemblies, while being required to be approved by the same agencies as the double check valve assemblies, offer the ultimate mechanical device for backflow protection. If one or both checks should fail to close tightly because of mechanical failure or foreign matter blockage in a double check valve, this failure would go unnoticed and backflow could occur. An "RP" assembly offers the additional feature of a visual notification of check failure (or blockage) by complete discharge to atmosphere through a pressure differential relief valve located between the two check valves. This discharge prevents the possibility of any water flowing back into the domestic supply should both check valves foul at the same time.

Health officials and water agencies are today commonly required to establish and enforce thorough cross connection ordinances. These ordinances should recognize fire protection systems as "water upon demand" piping systems. These necessary systems should be protected in the same manner as other potential sources of contamination to our potable water supply.

LIME IN WATER TREATMENT

by

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Myron Lahola, Operations Supervisor
City of Edmonton

INTRODUCTION

This presentation covers Lime in Water Treatment. However, it will not deal with the chemical reactions, which of course are well known, but with the physical handling equipment of the two major lime systems. The slide presentation will start out by describing how lime is manufactured and follow through with how lime is delivered and used.

Briefly, one of the purposes of Lime in Water Treatment is to soften by removing bicarbonates. Lime is also used for pH control. Additional benefits are: Heavy Metal precipitation (at high pH) and Purification - pathogen destruction (at high pH). The addition of alum or iron salts with lime also helps reduce turbidity. Dolomitic lime is used as a silica remover.

SLIDE PRESENTATION

Slide No.

1. Lime in Water Treatment.

2. View of Kananaskis Alberta Company townsite, valley and Steel Brothers lime and calcium plant located approximately 85 miles West of Calgary on No. 1A highway.

3. Old lime kiln (circa 1880s) near our present quarry and plant.

4. View of quarry approximately 14 km from plant site where lime stone is blasted in 90,000t lots. Poor quality rock and overburden is segregated and high calcium rock is trucked to our plant for processing. The estimated life of the quarry is 50 to 75 years depending on demand. The quarry is approximately one mile in length.

5. Shot rock is broken down by drop ball to manageable size.

6. It is then loaded by front-end loader into tandem trucks.

7. And transported over the No. 1A highway 14 km to our Kananaskis plant for further processing.

8.& 9. Raw rock is dumped into a jaw crusher where it is further reduced size.

10. The crushed limestone is then screened to various sizes.

11. And stock piled for use as needed.

12. 2 1/2" x 3/4" stone is fed by conveyor to the preheater.

13. The Preheater, which is like a large chimney where the exhaust gases from the kiln are captured, preheats the limestone to start the calcining process. This is to take advantage of the heated exhaust gases and greatly improves the energy efficiency of our calcining operations.

14. The limestone then enters the kiln where it travels by gravity from the "cold" end to the "hot zone." The internal temperature ranges from 1800^{0} to 2700^{0}F and is constantly monitored and controlled by our technicians.

15. After the limestone is fully calcined = all the CO_2 driven off = the resultant product, Quicklime, is captured in the cooling chambers under the hot zone end of the kiln.

16. Quicklime pebbles of various sizes are then conveyed to screens for specific sizing and storage in bulk bins to await shipment to bulk customers, or for bagging and reshipment.

17. From quicklime, CaO, we produce hydrate lime, $CaOH_2$. This is done in a hydrating machine pictured here. Quicklime is fed into a large mill and water added at a specified rate.

18. Water addition and other mechanical functions are monitored by this electrical panel. The result is a dry powder, hydrated lime, which contains approximately 25% H_2O.

19. Because the raw quicklime fed to the hydrator contains some inerts above 30 mesh size, the resultant hydrate product is fed through a pulverizing mill to reduce the final size to acceptable limits.

20. Then the finished product goes as well to bulk storage bins to await disposition. Our plant has over 2500 tonnes of bulk storage for lime products.

21. Bulk lime can be shipped by rail or truck from our storage bins.

22. We have our own new truck scale just installed in 1985 at a cost of some $350,000.

23. Here is a review of the product types and sizes.
After crushing and screening we produce 3 sizes of limestone ($CaCO_3$).

    1. 2" x 3/4" - Kiln feed stone
    2. 3/4" x 3/8" - Kiln feed stone
    3. 3/8" Minus - Calcium plant feed stone

24. From limestone we make quicklime by heating or burning as shown above.

    First we make Quicklime - chemically calcium oxide. Then we take the quicklime and add water to produce hydrated lime - calcium hydroxide, a dry powder.

The main difference between these two products is:

Quicklime - (CaO) - is a highly caustic chemical which, in contact with water will slake or hydrate giving off extreme heat - 490 Btu - or up to 40° in 3 minutes.

Hydrated Lime - ($CaOH_2$) - already contains approximately 25% water in a dry form which the manufacturer has slaked or hydrated so there is no heat reaction when further water is added.

As you will notice from the graph there is also another type of lime called dolomitic lime which is very high in magnesium content. At present this product is not manufactured in Western Canada because of the very limited demand.

Now the presentation will follow the two different types of lime through the water softening process.

25. Hydrated lime is used in smaller water treatment plants such as the Anthony Henday Water Treatment plant near Innisfail, Alberta.

26. Hydrated lime is delivered by pneumatic tank truck and blown by air into bulk storage silos located directly over the metering and slurrying equipment.

27. Bulk storage bins are equipped with air hammers and air injection to assist the flow of lime into the system.

28. Lime feeds by gravity into the metering system which introduces a predetermined amount of hydrated lime into the slurry tank.

29. In the slurry tank the hydrated lime is mixed with water up to consistency of 20% calcium oxide into a milk of lime. Grit settles to the bottom of the tank where it is removed manually from time to time.

30. Milk of lime is then transported by pumps to the clarifier. Notice the redundancy. The lines are normally set up in duplicate to allow for regular maintenance and descaling.

31. Milk of lime is pumped into the clarifier at a specified amount depending on the bicarbonate content of the raw water. Here it is mixed with the raw water and allowed time to react.

32. And this is the result, along with the addition of other chemicals. A clean, soft, potable water.

33. Thank you to Phil Richard Superintendent of the Anthony Henday plant for allowing us to use his plant as a model for hydrated lime use.

The slide presentation will now follow the use of quicklime through the City of Edmonton E.L. Smith Water Treatment plant.

This plant is a modern facility with the latest techniques for bulk quicklime use.

34. 1/4" minus Hi calcium high reactivity bulk quicklime is received by bulk pneumatic truck.

The E.L. Smith Plant lime facilities are all in duplicate to ensure continued service and maintenance.

35. Here is shown the two bulk lime silos - 300t capacity each.

36. Again bulk lime is delivered by pneumatic tank truck and is blown into the silo.

37. Adjacent to the silos is the pump house where the City has installed two - 600 CFM electric blowers to blow lime product from the tank truck into the silos.

38. Two electric blowers were installed to ensure reliable equipment would always be available.

All bulk silos must be equipped with adequate dust control equipment for obvious environmental reasons.

39. Notice here the two unloading pipes clearly marked lime and alum, as E.L. Smith also uses bulk alum.

40. Again, as with the hydrated system, the quicklime system is all vertical. The cone of the silos are directly over the additional lime equipment. Here we see the cone of the silo. Note again the air hammer and the air injection jets to assist the gravity feed of product. Air impact hammers can be activated as needed or set on a time schedule but one must be careful as too much use can cause compaction of the lime in the silo and reduce flow.

41. Inside the silo there is a metal casing with an inner rubber liner. Air is forced between the liner and the casing which causes a "para staltic" motion which massages lime down towards the feeder. This minimizes rat holing in the lime.

42. The air pressure and unit is activated automatically by the turning of the star valve.

This is an intermediate hopper with level sensors in it. When it reaches the low level the star valve is activated and feeds lime into the intermediate hopper.

43. Below it is the belt feeder which feeds the auger to the slaker. Feed rate of lime is determined by the hardness of the water to be treated and can be programmed in at the feeder or remotely from the control room.

44. This is the auger system from the feeder to the slaker which is located below the feeder. It is important to have complete vertical system ie. storage silo above feeder, above slaker to eliminate any lateral movement of lime. Any attempt to move lime horizontally normally creates difficulties.

45. Please note the left hand side of the slide shows a venting pipe where hot moist air created by slaking the lime is vented out so that it does not back up into the feed auger and cause incoming dry lime to stick and gum up. This was an innovative design developed by the City of Edmonton personnel and added to all their slakers.

Wallace & Tiernan - 1800 kg per hour. There are four slakers at E.L. Smith - two under each silo, one is a redundancy backup. They are alternated in use once a week. While one is down it is completely washed out and inspected. This has significantly reduced breakdowns.

46. This is another view of the slaker. Note the chain and flight grit removal system. The grit is ejected into a container and is removed by hand. This is the typical method of removing grit. Operators tell us there is about a 5 gallon pail every 12 hours when using Steels Hi Cal quicklime.

Wallace & Tiernan paste slakers operate best between 170 - 190°F. There is a torque valve on the water supply. The torque is set to maintain a peanut butter paste consistency and water is added by the valve to maintain this level. The lime paste is diluted by water at the discharge end of the slaker to an approximate consistency of 10 - 15% lime to water ratio.

47. The slurry is then transported by gravity into a slurry holding tank. This ensures the lime is fully slaked. The boxes adjacent to the tanks are level sensors that determine the level of slurry in the tank. Between the tanks are the feed pumps.

48. Lime slurry is pumped from here to the clarifier through heavy wall 3" pipes. Pipes are pigged out on a daily basis to reduce firring or scaling. The entire system is acidized from the slaker to the clarifier once a year.

49. This is Myron Lahola, past Operations Supervisor and Milt Tkachuk Lab. scientist at E.L. Smith.

50. Lime slurry then enters rapid mix area of solid contact "graver" reactor (clarifier). The lime then precipitates the bicarbonate from the water.

51. This is the finished product - clear soft water. Of course other chemicals are used for other purposes to complete the process.

52. This is a Pig Station where the pig is retrieved from the feed pipe.

53. This pig is a City of Edmonton inhouse design.

54. This is the sending station.

55. This is the computer board which shows bin levels, which system is operating and relative location of equipment. It's a diagram of the entire lime handling system.

56. Here we have the control panel where you select which equipment is to operate and in which mode (manual or auto). This is the breaker box for all equipment and alarm anunciator.

57. This is the main control room which oversees the operation of the entire plant. Lime functions can be monitored from here as well.

CONCLUSION

We have just touched on the highlights of lime in water treatment. It is important to note that either hydrate or quicklime can be handled cleanly and efficiently when the systems are properly equipped and designed. A number of brochures with some lime information are available from the author.

ACKNOWLEDGEMENTS

Thank you again to Myron of the City of Edmonton Water Treatment Department and Phil Richard of the Anthony Henday Regional Water Treatment plant.

WATER TREATMENT PLANT TROUBLE SHOOTING

by

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This is a very large subject indeed because of obvious variations from plant to plant in so many things: raw water quality, type of process, equipment and chemicals. Problems may be due to design, operation, equipment failure, etc. and combinations of all of these.

Accordingly we chose to discuss trouble shooting at one particular water treatment plant, the identity of which we prefer not to disclose. This plant, as you will see, had quite a number of problems. Problems are obviously never the same from one plant to another but some things discussed may well relate to your own water plant.

The schematic flow for the plant is shown in Figure 1, with raw water pumped from a nearby river. Flow is to a solids contact clarifier, referred to hereafter as an S.C. unit, which ran as a cold lime softener during high hardness, low turbidity periods. Chemicals fed are chlorine, lime, soda ash, alum and polymer. During high turbidity and suspended solids periods, like spring break up, the unit would be switched to a straight clarification mode using chlorine, alum and polymer.

When operating as a cold lime softener, sulfuric acid is added to the effluent to stabilize the water by lowering the pH. The flow went by gravity to an open top concrete tank which served as a filter pump suction tank and as a fire water storage reserve.

Water is pumped from this tank to eight horizontal pressure filters in parallel. Filter media are coal and sand with base gravel layering. The filters are equipped with an air scour system. Filtered water flows to a common filtered water header.

Backwash of a filter is initiated by P with automatic sequencing of the valves for isolation, drain down and air scour, with hydraulic backwash from the common filtered water header.

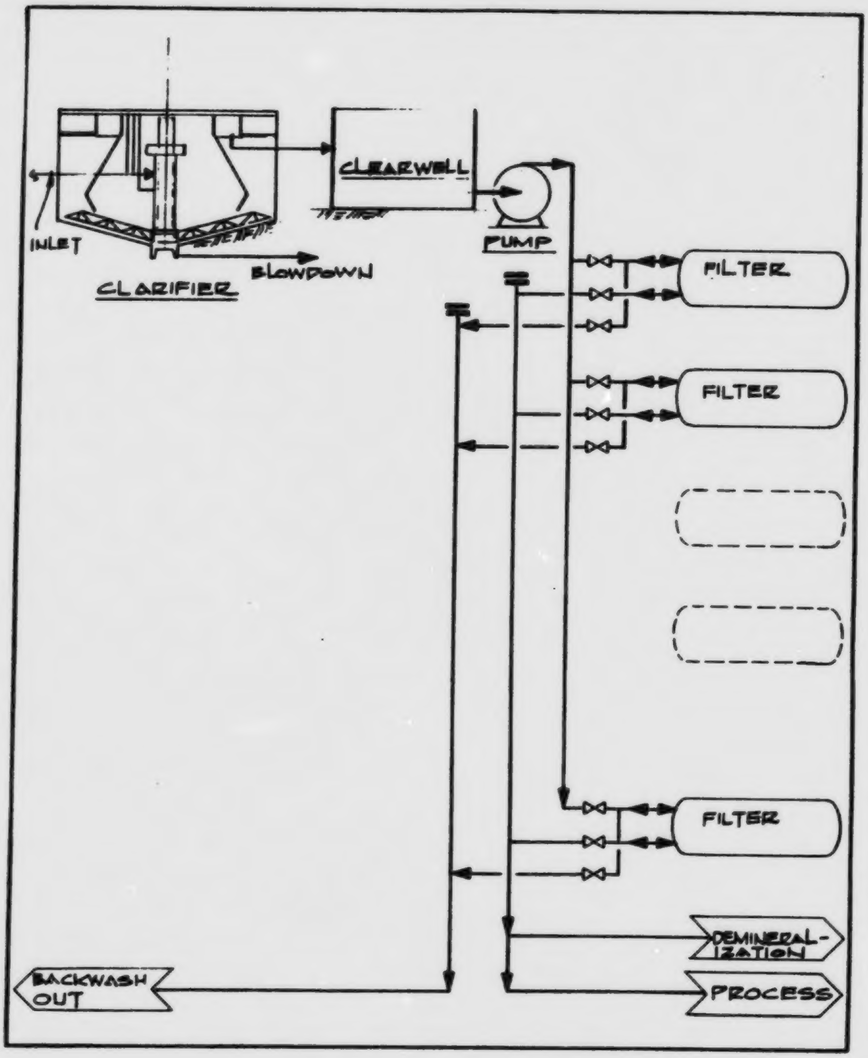
Dirty backwash water is collected in a backwash reclaim tank and pumped back slowly to the S.C. unit inlet.

We should mention also, that there was return stream from process of warm water added to the S.C. unit inlet to get some heat into the raw water.

Filtered water was then split for various uses: to boiler water pretreatment demineralizers, to process, to pump seal water, potable water etc.

Basically a pretty straightforward process. The plant had started up in late fall running in the lime softening mode. During 4 1/2 months of operation serious problems had occurred. S.C. unit performance was poor with excessive carry over and poor softening results. The filters were plugged up with lime

Figure 1 - Flow Diagram



sludge, the air scour system was inoperable, the filtered water contained sand and was turbid. The demineralizers had plugged up with sand and lime sludge and, even before plugging occurred, could not keep up with higher than design hardness. Sand in the pump gland system resulted in big problems. The lime and soda ash feed systems had serious line plugging problems, so softening control was poor, and attempts to alleviate this problem had made matters worse instead of better. These water plant problems had shut most of the process plant down. To add to the general gloom, the waste treatment primary clarifier and sludge thickener were inoperable, but we will not discuss waste treatment trouble shooting here nor demineralizer trouble shooting.

The first thing to do in trouble shooting is to take a look at the basic design parameters of the process units, starting at the front end. Is there any basic reason why the plant cannot produce to design flows and effluent quality? Surprisingly often there are design flaws of many kinds.

Solids Contact Unit

Design normal flow Qn = 16000 USGPM
Design maximum flow Qmax = 18750 USGPM
Size = 144 ft. dia. x 21 ft. SWD
Volumetric Capacity = 2,785,000 USG
Net clarification area = 15850 sq. ft.

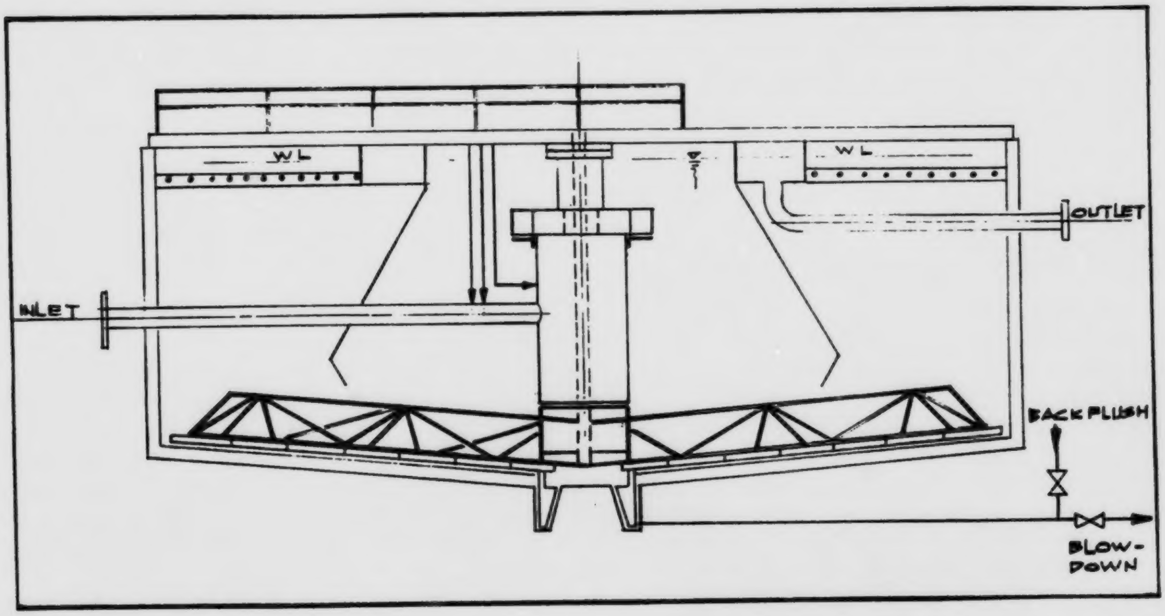
	Rise Rate (USGPM/sq ft)	Detention (Minutes)
Qn	1.0	174
Qmax	1.18	148

Effective detention time in rapid mix and flocculation zone = 20 minutes approximately at Qn.

There is nothing evident in these design parameters to prevent good operation. The rise rates with no tube settlers are about as high as one would want to go but on lime softening should be okay. When the unit has to run in the clarification mode with cold water, operation without excessive carry over of floc would be tricky. Compensations are that the warm water return should keep the clarifier temperature about 40 - 45°F which will help a lot, and the detention times are fine.

Figure 2 shows the internals of this particular S.C. unit. Raw water and chemicals flow into the centre draft tube. Flow is then upward through a rotating turbine which causes a flash mixing action, then down through a baffled flocculation zone and upwardly through the clarification zone. Clarified water is taken off evenly through radial pick-up launders, flows to a central launder and leaves via the outlet pipe.

Figure 2 - Clarifier



Floc settles out of the flocculation zone and the clarification zone and is raked inwardly. The turbine draft tube extends well down towards the floor of the unit, leaving in this unit a 2 1/2 ft. gap. The upwardly turbine-induced flow pulls most of the raked-in sludge back up to mix with the raw water and chemicals. This induced sludge recycle flow is variable with turbine speed but is very large, typically 5 - 7 x Qn. Sludge which is too heavy to resuspend drops into a sludge collection sump which is slowly stirred by sludge cone scrapers to cause additional thickening before being removed through the blow down line.

High volume sludge recirculation is the absolute heart of a S.C. unit. Calcium carbonate precipitated from the lime softening reaction (and magnesium hydroxide also to some extent) tends markedly to supersaturation, and this function is vitally important. If the supersaturation is not broken in the unit, then the feed of sulfuric acid (or carbon dioxide as is commonly used) to the S.C. unit effluent will react to break it but the effluent hardness will be higher than it need be. Also, if control of the pH adjustment system is poor, the effects of supersaturated calcium carbonate on downstream equipment is devastating. The filter media acts as a very good nucleator and encrustation of the media will occur rapidly. Some of the colloidal calcium carbonate will pass right through the filter and end up in the ion exchange system and also encrusting pumps and lines.

Hardness in supersaturation is often referred to as "hidden hardness". The reagents used in standard hardness testing react quickly with ionic hardness but slowly with colloidal hardness. If you see a difference between a standard hardness test and an acidified hardness test (on filtered samples) you know supersaturated hardness is present.

A passing note on the importance of sludge recirculation in a S.C. unit operating as an alum clarifier, though this particular unit had not yet operated as an alum clarifier. The function of sludge recirculation is to provide agglomerating nuclei for aluminum hydroxide to build on. Alum floc in cold water tends to be very light and delicate and carries over violently if not properly flocculated. It is important in this operation that a lot of sludge be recirculated, but gently, i.e. at low turbine tip speeds to avoid shearing the floc.

Observation of the S.C. unit current operation and a quick review of past performance and operation led to a rapid conclusion as to problem causes and cures.

Sludge blowdown had not been sufficient, so that sludge too heavy to be recirculated had piled up in the central part of the unit and had totally blocked off the bottom of the draft tube so sludge recirculation was non-existent. This situation occurs through a misunderstanding of the mechanism and is a vicious circle type of thing. Because of insufficient blowdown, sludge recirculation starts to become impaired and performance starts to deteriorate. The operator notes the deterioration in sludge quality in the flocculation zone, i.e. the sludge becomes thinner and thinner. Thinking the problem to be one of too much blowdown with a depletion of sludge inventory, the operator reduces blowdown and conditions quickly and steadily get worse and worse. (This kind of problem can occur very fast in units where there is a lot of silt and fine sand in the raw water, which is a by-the-way as the raw water here was relatively free of silt and sand).

This unit had had practically no blowdown at all for nearly a month and the rake torque indicator was reading 15 - 20% of full load torque. Floc zone sludge was very thin and hidden hardness was very high in samples ahead of the sulfuric acid feed point. It was obvious that the bottom of the draft tube was blocked off. Immediately we started a back flush program to free up the sludge and a blowdown program to get the excess sludge out and in several days the unit started working beautifully with no hidden hardness and less than 5ppm turbidity ineffluent and rake torque of 2 - 3%.

Looking at the records we found that there had been almost a two month period when no sulfuric acid had been fed because of a problem in the acid feed system that had received no priority to fix. The system was operational when we arrived but one can imagine, in view of the above discussion, what a disastrous effect this outage had on the filters and demineralizer system.

A note again in passing re S.C. unit operation. Too much blowdown will deplete the sludge inventory to the detriment of good operation. Blowdown as much sludge in a day as you made the previous day. Proper operation really requires a good knowledge of the particular unit and an awareness of these factors already discussed.

There were some other problems with the unit, or rather factors that were assumed by the operators to have contributed in major ways to the poor operation. We concluded that these factors were relatively minor, for example:

The area is quite windy and the clarifier had only 8" or so of free board so the surface of the clarifier would get fairly wavey. It would have been a good thing to provide some wind protection with a peripheral 4 ft. or so high wall of fibreglass panels for instance, but the performance of the clarifier would be improved only very marginally, not dramatically.

The volume and temperature of the heated water stream from process back to the S.C. unit inlet was quite variable and tended to swing the temperature in the reaction zone back and forth through 38°F, the point of maximum density of water. This is not a very good thing as it gives rise to thermal and density currents but we would expect this to have just a marginal effect and not a devastating one.

Use of alum as a coagulant aid in lime softening is common but not highly recommended. Alum has an acid reaction and converts some of the calcium and magnesium bicarbonate to permanent hardness. Sodium aluminate is a better choice. So little alum was used, approximately 10 mg/l, that we concluded the effect was negligible.

Some operators questioned whether the polymer used was doing the job and wanted to embark on a major jar testing program to pick a better one, blaming the existing polymer for all the problems. As we have seen, it was not the fault of the polymer at all. In jar tests it worked not badly. Perhaps another would work better but our experience is that unless the polymer is really poor in a jar test, another one will not work miracles.

Others felt that tube settlers should have been installed initially. (Perhaps it would be a good idea when they try to run a clarification mode.) However, tube settlers will not stop hidden hardness or particulates that are very fine and poorly coagulated. Under the circumstances described, tube settlers would make no difference one way or the other.

CHEMICAL FEED SYSTEMS

Problems here contributed to poor operation. The loss of the acid feed system for nearly two months was disastrous. In other plants a poorly designed recarbonation system can be disastrous also. The importance of pH adjustment following lime softening and good control of pH cannot be overemphasized. Anyway, the acid feed system was going again and working well.

The lime system had been very troublesome, with rapid plugging of the lime feed lines to the S.C. unit. This resulted in the erratic feed of lime and poor control over the softening reaction plus, of course, a real maintenance headache.

Slaked lime was prepared elsewhere in the process plant and pumped at 5% by weight slurry over to a lime feed tank at the water treatment plant, then pumped to the S.C. unit proportional to flow.

This seems simple enough and should have worked fine except the line to the S.C. unit was 2 1/2" Ø and normal flows of 5% slurry would result in a line velocity of 1.4 fps. The designer had figured on a minimum velocity required to keep lime slurried of 2.5 fps, so a dilution water line was installed to the pump discharge to keep the line velocity to 2.5 fps minimum. Rapid plugging of the line occurred. The cure for this was assumed to be a higher velocity so the volume of dilution water was raised to maintain 5 fps velocity and plugging got worse.

Overlooked here, and incidentally on many other lime feed systems, is the fact that lime slurry reacts rapidly with hardness components in the dilution water to lay down scale in very short order. The more dilution water used, the more scale results. Even if the water is zeolite softened, the lime will react with sodium bicarbonate to deposit scale. Condensate or demineralized water will not react but seldom are there any of those things around and demineralized water is too expensive anyway. Water that has already been lime softened, as was used here, is not the answer because the high concentration of lime will push the softening reactions further to completion.

We simply shut off the dilution water and if 1.4 fps velocity is not quite enough to keep the lime slurried they would have to repipe in a smaller line.

The soda ash system had a problem as well. Ash was fed via a volumetric feeder dropping into an agitated solution tank and pumped to the S.C. unit. Solution water is added to the solution tank automatically on level control. Soda ash reacts with hardness components in the solution water to precipitate scale throughout. Correcting this is not quite so easy. Required is a second tank or a much larger dilution tank so that softening reactions can complete and the solution is stable before trying to pump it.

Note that some other chemicals used in water treatment are reactive with dilution or solution water, for example, HTH (which contains a lot of excess lime), caustic soda and sodium aluminate (highly caustic). Special care must be taken to stabilize these chemical solutions before pumping.

FILTERS

The filters depicted in Figure 3 and view and Figure 4 side view were in a terrible mess. There was a lot of grain encrustation, layers of lime sludge and serious mud ball formation. The filtered water was sandy and turbid (little difference in turbidity in or out). The air scour system was inoperable.

Figure 3
Sand Filter Tank Cross Connection

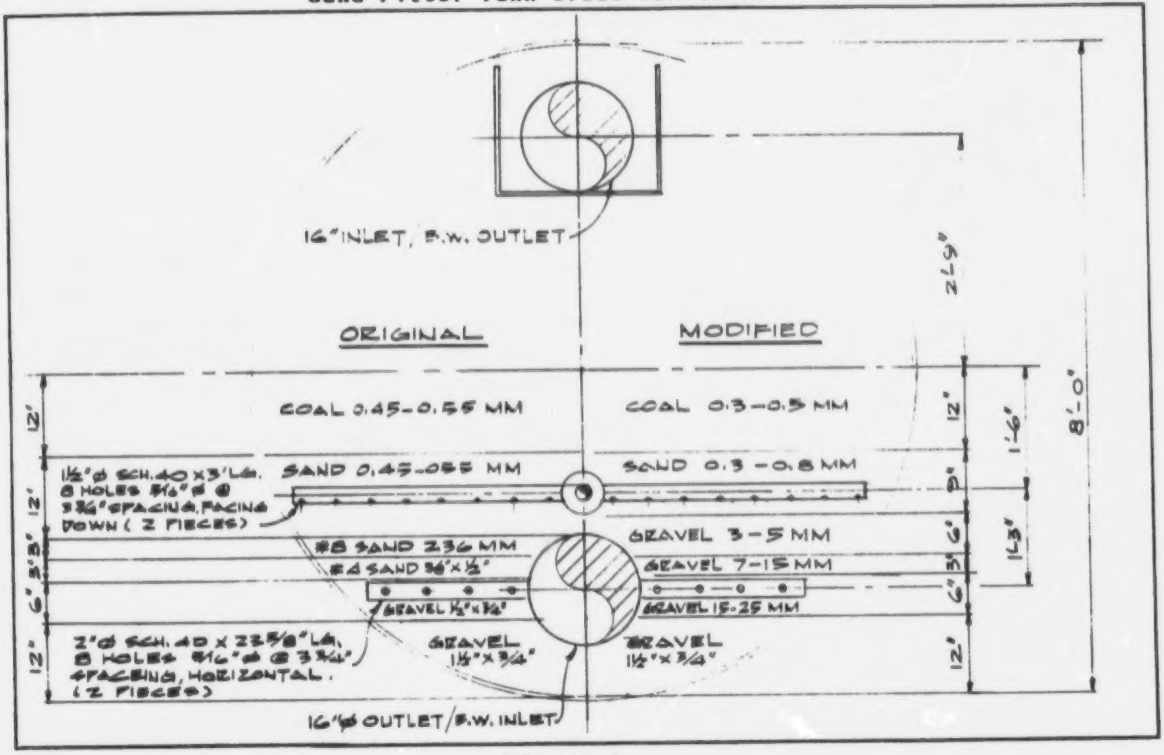
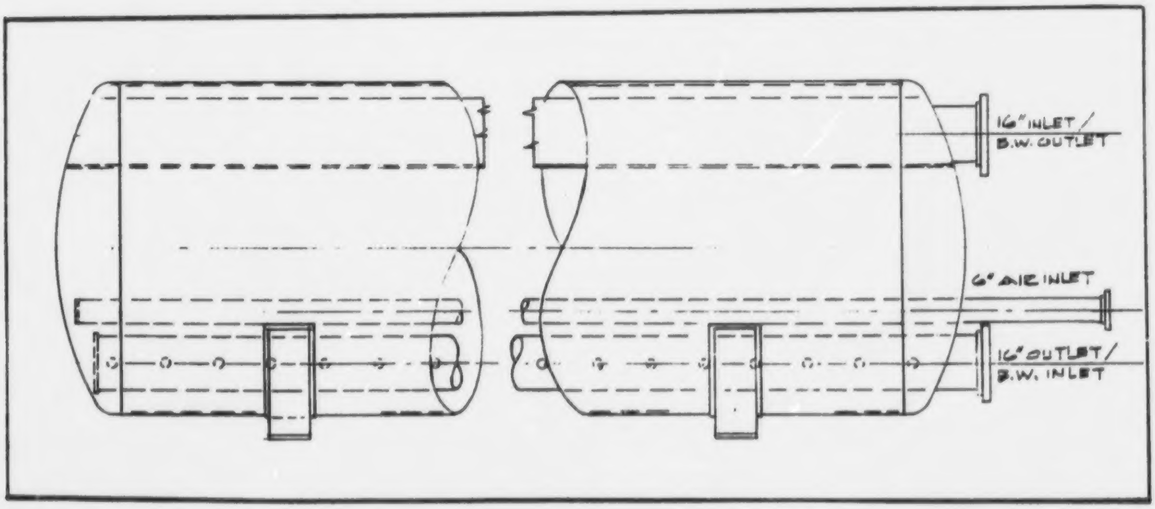

16"INLET, B.W. OUTLET
2'-9"
ORIGINAL
MODIFIED
COAL 0.45-0.55 MM
COAL 0.3-0.5 MM
SAND 0.45-0.55 MM
SAND 0.3-0.8 MM
1½"ø SCH.40 x 3' LG. 8 HOLES 5/16"ø @ 3¾" SPACING, FACING DOWN (2 PIECES)
#8 SAND 2.36 MM
#4 SAND ⅜"x½"
GRAVEL ½"x¾"
GRAVEL 3-5 MM
GRAVEL 7-15 MM
GRAVEL 15-25 MM
2"ø SCH. 40 x 23⅝" LG. 8 HOLES 5/16"ø @ 3¾" SPACING, HORIZONTAL. (2 PIECES)
GRAVEL 1½"x¾"
GRAVEL 1½"x¾"
16"ø OUTLET/B.W. INLET
12"
12"
6"
3"
3"
12"
12"
9"
6"
3"
6"
12"
1'-6"
1'-3"
8'-0"

Figure 4
Sand Filter Tank - Side Elevation


16"INLET / B.W. OUTLET
6" AIR INLET
16" OUTLET / B.W. INLET

Reasoning had been that the S.C. unit was to blame. If it had been operating well there never would have been all the sludge to the filters. Now that the S.C. unit was running well, so would the filters. Unfortunately, this was not to be.

Filter Design Parameters

8 horizontal pressure filters, 8 ft.Ø, 50 ft. long.
Filtration area at C/L 392 sq. ft. each.

		Filt.Rate 8 online USGPM/sq. ft.	Filt.Rate 7 online USGPM/sq. ft.
Design normal flow Qn:	15950 USGPM	5.09	5.8
Design maximum flow Qmax:	19360 USGPM	6.17	7.06

Scour air design 1200 cfm = 3.06 cfm/sq. ft.
Backwash rate 4994 USGPM/filter = 12.74 USGPM/sq. ft.

This design looks okay. Filtration rates are a bit on the high side but no great cause for concern. The air scour rate is fine, typically on filters 3 - 4 cfm/sq. ft. The backwash rates are fine for adequate fluidization even at summer temperatures. (Backwash rates that will give 24 - 25% fluidization have proven to be all that is needed. Not 50% as in some older designs.

So the filter process parameters are fine. How then, could all these problems have arisen? We opened a filter to have a look. The top of the bed was very uneven, high at the edges and high in the middle with layered sludge accumulations along the sides and middle. The top elevation of the media at the far end of the filter was several inches lower than at the inlet end. Mud ball formation was prevalent along the sides and in the middle. Much mixing of sand with coal was evident.

This tells you right away that there is a very serious problem with the underdrain design. Mud balling is a certain sign of uneven rise velocities on backwash as is an uneven surface cross-section. Service flow will then channel through those depressed areas and service rates, which were a shade high anyway, will obviously be perhaps doubled i.e. up to 10-11 USGPM/sq. ft. of "effective" surface area and filtration would be expected to be poor.

Sand breakthrough in the filtered water is a certain sign that the gravel bedding either is too coarse to retain fine sand, but more usually means that the gravel layering has been disrupted.

The difference in bed elevation from one end to the other is a definite sign of uneven backwash distribution longitudinally.

As far as the air scour system is concerned, note in Figure 3 that the air scour laterals are located in the fine filter sand and have 5/16" holes in them. One would surely expect that when a filter is operating at 100 psi pressure with no pressure in the air scour system it will certainly push sand into the air scour laterals and header and that is exactly what happened. The air cannot move wet sand back out so the air scour system was totally plugged with packed sand.

Had the air scour system been operable it would no doubt have helped to prevent mud balling, but not totally. As the laterals are on 15" centres, there will be a markedly poor distribution of the air and a rolling action to most of the bed, rather than an even scouring action.

Looking again at Figure 3, which is to scale, the original layering is shown on the left. Size designations are per the filter supplier.

Note that the backwash laterals are short, only 2 ft. long. There is very little gravel layering to spread the backwash flow and it is pretty obvious that on a backwash the side areas of the filter and in the middle will be "dead" spots or areas of lower flow rates than above the laterals. Above the laterals backwash, rates will be high and will tend to throw media over to the sides and to the centre, which, being above the 16" backwash header, will be a "dead" area also.

Note also that the laterals have holes along the sides. [Lateral holes should always be downward or staggered 15° from vertically down.] Backwash jet velocities out of these lateral orifices are very high (18 fps), high enough certainly to displace the gravel little by little, particularly as these laterals are on 15" centres. The tendency is to push gravel away from the lateral and little by little the next layer comes down and the next until sand can start coming through during service flow. Because the laterals are so widely spaced and there is so little gravel to spread the flow, the rise rates above the laterals will be very high. Recall that design backwash rates are 12.74 USGPM/sq. ft. at centre line at 4994 USGPM. If flow spread was even over the laterals, the backwash rate in that area would be 26.5 USGPM/sq. ft. As flow spread will not be good, the area above the laterals will be seeing a higher backwash rate than that. For example, by simple particulate geometry we would expect a point source flow to spread in diameter by 1" per inch of gravel. Starting then from a 2" diameter lateral, we would expect flow spread 6" above the lateral to 8" wide. Each lateral on backwash at design rate runs 71 USGPM/lateral. If we take that flow 6" above the lateral as passing through an area 8" wide x 2 ft. long, the rate is 53 USGPM/sq. ft. which is a rate which will readily fluidize gravel in the 3 mm size range. So we would expect the 3" layer in Figure 3, designated 2.36 mm, to fluidize above the laterals. If you have a design where areas of the barrier gravel will fluidize, there is certain to be serious trouble ahead with sand in the filtered water.

The longitudinal variation in bed level indicates that the header lateral system does not distribute backwash flow evenly. Looking at the design, this is to be expected. Calculations show that at the far end, end laterals will have a flow 70.5% higher than the first inlet end laterals. A 5% variation is good design practise; 70.5% is totally unacceptable.

The reason for this effect is due to a variation in lateral inlet flow coefficients because of velocity changes along the header. Obviously the lateral inlets at the front end of the filter see a higher velocity across them than those at the far end. Simply stated is that the water finds it easier to turn into a lateral as the velocity slows down. The far end of the filter will backwash harder than the front end and the filter service flow will try and channel through the back end. Media will gradually shift, resulting in the difference in bed elevation noted.

To compound all these design problems was the backwash system itself. After a filter was taken out for backwash, the sequence was to drain several feet of water out, to avoid air blowing the media into the backwash troughs, then air scouring and then the 16" outlet/backwash inlet would open to a set point to admit backwash water from the main filtered water header. As this header ran at 100 psi, and the valve had no rate of opening control, and the filter itself at atmospheric, one can imagine the violent disruptive slug of backwash water the filter would see.

When we arrived on site, some remedial measures were underway. This consisted of taking a filter out of service, sluicing the media out, cutting off the sand-plugged laterals, flushing sand from the air scour header, welding in new laterals to the same design but wrapped with stainless steel mesh to prevent sand ingress. Then the filter was rebedded with a modified gradation as shown in Figure 3. This new gradation was per consultation with the filter vendor. All media was on site to redo all 8 filters as described above.

One problem with the media becomes evident; the new recommended "barrier" gravel size 3 - 5 mm is too coarse to block off fine silica sand on the service flow. The filters re-done this way would have all the effects of poor underdrain design previously discussed plus an added tendency to transmit sand into the filtered water. All of the gravel supplied was 100% crushed which makes it easier to disrupt than rounder material. The only good thing was that the air scour system would no longer plug with sand.

Despite our strong trepidations and pleadings, four filters were rebuilt to the modified design, put on line and immediately began to pass sand into the filtered water. That was a costly exercise in futility. They did this really to avoid voiding the manufacturers process warranty by deviating from the manufacturers recommendations.

The upshot was a major rebuild of all eight filters. Vertical tees off the main outlet/backwash header were installed to get the laterals higher to minimize dead space at centre line. The number of laterals was doubled and reduced in diameter down the length of the filter to try and even out longitudinal maldistribution. Orientation of lateral holes was downward to dissipate velocity and the laterals were wrapped with stainless steel mesh to prevent sand ingress. We didn't think that was really necessary but it cost little to do relative to the very large cost of the rebuild. Gravelling was revised to proper gradations and thickness. This resulted in the top of the media being about 9" above the centre line but that is no problem unless one goes too high. The backwash inlet valves were equipped with slow opening snubbers. The old laterals were cut off and the connections closed with plates. The air scour header had to be raised to make room for the elevated bottom header laterals. The laterals on the air scour header were doubled in number. Properly compatible fine sand and filter anthracite were used (neither the original or modified coal/sand combinations shown on Figure 3 are compatible).

The filters now run fine, as does the rest of the plant.

TROUBLESHOOTING ACTIVATED SLUDGE WITH A MICROSCOPE

by

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HISTORY OF THE ACTIVATED SLUDGE PROCESS

The activated sludge process is the most efficient biological wastewater treatment process. The original experimentation was performed in 1912 by Dr. H. W. Clark. He determined that sewage could be clarified by mixing it with sewage sludge, agitating it by bubbling air through the mixture for approximately twenty-four hours, and then allowing the resultant liquor to settle under quiescent conditions.

The quantitative relationships of the process, such as the proper proportion of sludge to raw sewage and the required aeration and settling times, were determined experimentally on a pilot scale, notably by I. E. Ardern and W. T. Lockett (Cramer, 1931). They showed that both a sludge/raw sewage ratio of 0.25 and six hours aeration were sufficient to produce clarification. Their initial design, which employed long narrow aeration tanks, became the standard from 1914 to the present. Ardern and Lockett are best remembered today for their coining of the phrase "activated sludge".

In older sewage treatment processes, such as trickling filters, the presence of nitrates in the effluent was used to determine the degree of treatment obtained. As the nitrification of the effluent was thought to be so important, it is interesting to note the definition of the activated sludge process suggested in 1916 by the American Society of Civil Engineers:

> "Activated sludge treatment may be defined as a biochemical process by which the purification of sewage is accomplished by passing it through tanks, in which sewage sludge is artificially agitated and intimately mixed with sewage and is supplied with the requisite oxygen for the optimum development of countless nitrifying organisms incorporated in and adhering to the sludge, the final settlement of which causes a distinct clarification of the oxidized sewage" (Cramer, 1931)

Since that time, practical experience with the process has shown that nitrification is costly, and often detrimental to the performance.

The first changes to the activated sludge process came when McKinney (1956) recommended the complete mix aeration tank in response to studies at the Massachusetts Institute of Technology. These studies noted that the conventional activated sludge process (that is, plug flow) was not properly designed from a biological treatment standpoint. It was found that the bacteria were in a feed/starve cycle; one in which bacteria are alternately fed in the aeration process and later starved in the clarifier. This cycle, in turn, made the process susceptible to upset, as it was never in equilibrium with the organic load imposed on the microorganisms. The net result caused problems when the load varied significantly from the average.

Since that time, other modifications to the activated sludge process have been made, including: tapered feed; tapered aeration; and contact stabilization. Each of these modifications was made in an attempt to reduce the sensitivity of the process to varying loading rates and wastewater strengths. As a result, a proper environment for the maintenance of the biomass was provided.

HISTORY OF THE USE OF MICROBIOLOGICAL INDICATORS

Buswell and Long (1923) were the first to report on the microbiological succession of organisms in activated sludge which, in part, can be seen in Figure 1. From their observations, they proposed a theory of the activated sludge process, but it had little impact on the engineers who designed the systems (McKinney, 1968).

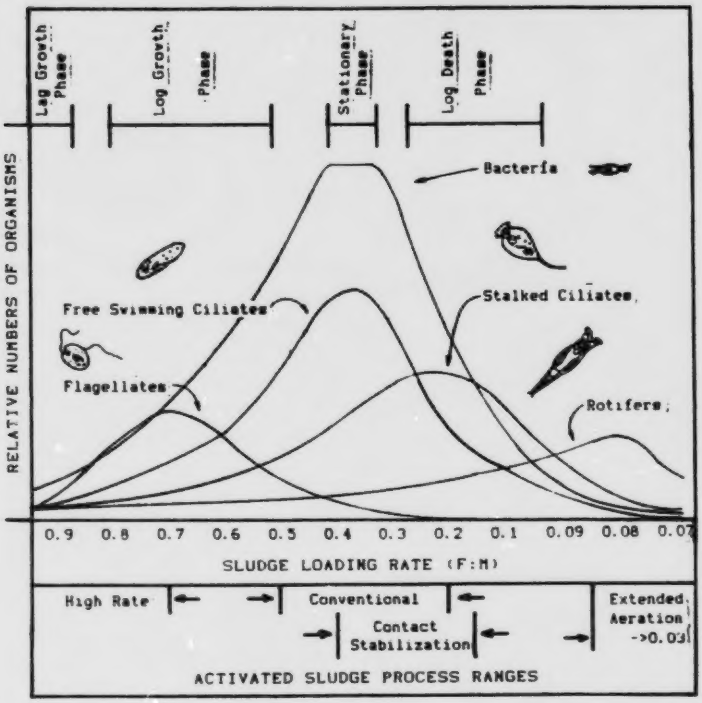
It was Butterfield (1935) who isolated the first floc-forming bacterium in activated sludge. The observation that *Zoogloea ramigera* was the primary bacterium for forming floc, and was capable of stabilizing organic matter in the same manner as activated sludge, established the bacterial basis of the process.

As protozoa were so obviously part of the process and were easy to observe, efforts were made to determine their role. Cramer (1931) suggested that the process could not function properly without protozoa and further noted that *Vorticella* could be used as an indicator of a well-operating process. Heukelekian (1931) and Pillai and Subrahmanyan (1942, 1944) believed that protozoa were the primary agents of stabilization and that bacteria played a minor role. It was not until studies by McKinney and Gram (1956) that the dynamic relationship between protozoa and bacteria was explained. They determined the role that each played in the process and later confirmed the potential for the use of protozoa as indicators.

Curds and Cockburn (1970a, 1970b) undertook studies of the fauna in British activated sludge plants. They determined that effluents of high quality contained a wide variety of ciliate protozoan species while low quality effluents either had none, or only a few species. They were further able to correlate protozoa species with both the food to microorganism ratio (F:M), and the biochemical oxygen demand (BOD) of the secondary effluent, thus leading to a basis for troubleshooting the process.

The usefulness of using protozoa for troubleshooting the activated sludge process has generally been ignored in favour of what have become standard tests and mathematical analyses of the hydraulic and other physical parameters. This may be a result of the perceived difficulty in identifying the protozoa in the process. Engineers are normally taught how to do the mathematical analysis, but not how to determine the significance of the protozoan population structure.

FIGURE 1
Growth Phase Graph



ROLE OF PROTOZOA IN ACTIVATED SLUDGE

Effluents from wastewater treatment plants should be transparent, hygienically safe, and free of soluble organic materials (Sudo and Aiba, 1984). The role of protozoa in the activated sludge process is to bring the effluent to this state. More specifically, protozoa:

1) enhance the flocculation of bacteria, thus aiding sedimentation;

2) remove dispersed bacteria by adsorption;

3) decrease the abundance of dispersed bacteria through predation;

4) maintain a healthy population of flocculated bacteria through predation;

5) provide bacteria with more "food of soluble substrate" by the reduction of the numbers of dispersed bacteria and through the direct uptake of suspended substrate; and

6) reduce the numbers of pathogenic microorganisms.

TROUBLESHOOTING WITH PROTOZOA

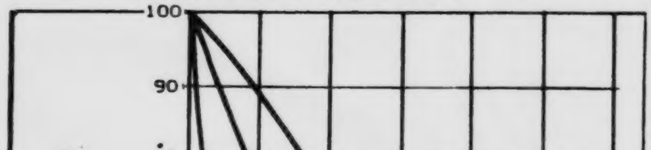
There are four basic types of activated sludge processes; conventional, contact stabilization, high rate, and extended aeration. Each of these processes operates in different phases of protozoan growth; and therefore, the types and diversity of the protozoa that live in those processes, under optimal operating conditions, will normally be different. Each of these processes is shown in Figure 1 in their approximate location on the standard growth phase chart which is based on the sludge retention time (SRT). As noted in Figure 1, all the protozoa of interest can be found in almost any type of sludge. The important factor is the relative predominance of one or more particular species.

In order to troubleshoot the process effectively, it should be recognized that not only are the abundance and species composition of protozoa important, but the shape and size of the sludge flocs, the length of the filamentous organism that is the primary component of the floc, the amount of colloidal material between the flocs, and the existence of any other organisms, such as nematodes, are also important.

For the operator who is learning to troubleshoot the activated sludge process with a microscope, it is useful to compare the microscope observations with the visual observations of a well-known test. The Thirty Minute Settling Test (SV30) has been chosen by the authors for such a comparison. The SV30 Test is performed by allowing a volume of activated sludge to settle in a graduated cylinder. Observations are made every five minutes for a thirty minute period. Information is recorded concerning the settling rate of the sludge (as shown in Figure 2) the volume of the settled sludge as a percent of the original volume, the colour and odour of the sludge, and the turbidity of the supernatant. From this information the operator is able to troubleshoot many process failures.

FIGURE 2

SV30 Settling Curves



A troubleshooting list, based on a comparison of SV30 observations and microscope observations, for various types of activated sludge processes and sludge conditions is contained in Table 1. This table was developed from the authors' field observations [Environment Canada, 1985; Environment Canada, 1986], and from the available literature. Users of this table should recognize that each process and each facility is unique and variations will occur. These variations are a result of many factors, including: types and volume of industrial waste contributions; diurnal flows and peaking factors; and process configuration. Operators should take detailed notes, comparing microscope observations to process efficiency and stability, until benchmarks are established for a specific facility.

TABLE 1
Troubleshooting List Using a Comparison Between SV30 and Microscopic Observations for Various Activated Sludge Types

SLUDGE TYPE	SV30 OBSERVATIONS	MICROSCOPE OBSERVATIONS	TROUBLESHOOTING
Normal (Conventional)	The sludge is chocolate brown in colour and has a slightly musty odour. The supernatant is clear with a well defined sludge/supernatant interface. Good settling characteristics - SVI 90 to 120. Properly aerated sludge usually stays down 2 to 3 hours if left in the cylinder.	Flocs are regularly shaped with little collodial material between them. Filaments may slightly protrude from the flocs. There is a balance between free swimming ciliates and stalked ciliates. Higher trophic levels may occur.	Normal Operation
Normal (Extended Aeration)	The sludge is chocolate brown to golden in colour and has a musty odour. The supernatant is slightly turbid with a light brown or golden colour and a well defined sludge/supernatant interface. Good to rapid settling characteristics - SVI 80 to 120. Properly aerated sludge usually stays down 2 to 4 hours if left in the cylinder.	Flocs are small and spherical with a slight amount of colloidal material between them. Large numbers of stalked and colonial ciliates are present. Nematodes and higher tropic levels may occur. Filaments do not normally extend beyond the edge of the floc. Suctoria may be observed.	Normal operation
Normal (High Rate)	The sludge is light brown to brown in colour and has a slightly musty odour. The supernatant is clear with a few straggler flocs and a well defined sludge/supernatant interface. Good to slow settling characteristics - SVI 100 to 150. Properly aerated sludge should stay down for 2 hours if left in the cylinder.	Flocs are large with ragged edges and little colloidal material between them. Filaments may protrude slightly from the flocs. Ciliated protozoa are predominant. Both flagellated and stalked specimens may be observed. Higher trophic levels are unlikely to occur.	Normal operation

Table 1 - continued:

SLUDGE TYPE	SV30 OBSERVATIONS	MICROSCOPE OBSERVATIONS	TROUBLESHOOTING
Normal (Contact Stabilization - Contact Zone)	The sludge is light to dark brown in colour and has a musty odour. The supernatant is generally clear with a few straggler flocs and a well defined sludge/supernatant interface. Good to slow settling characteristics - SVI 100 to 150. Properly aerated sludge should stay down for 2 hours if left in the cylinder.	Flocs are large with ragged edges and little colloidal material between them. Filaments may protrude slightly from the flocs. Flagellated, ciliated, and stalked ciliated protozoa should be present in equal portions. Higher trophic levels may also be seen.	Normal Operation
Billowing	SV30's are normal but the sludge is going over the weir of the secondary clarifier.	Microscopic observations should indicate normal operation if this is the first instance of billowing. There will be a population shift toward lower trophic levels if the problem has been occurring for awhile.	Hydraulic surges overloading the clarifiers.
Pinpoint or Ash Floc	Light brown to grey in colour; supernatant is turbid with pinpoint solids. Characterized by rapid settling - SVI 50 to 90. Gas bubbles may occur in the sludge after 1 hour.	Very small spherical flocs; no noticeable filaments extending beyond the edge of the flocs. Stalked ciliates nematodes, and rotifers predominate.	SRT is too long (i.e. there is too much biomass). Most likely to occur with an extended aeration process.
Anaerobic	Dark brown to black in colour; may settle well initially but quickly rises to surface. Smells of rotten eggs.	Flocs and microbiological populations may be normal for process type. Filaments are extended beyond the edge of floc. Flagellated and ciliated protozoa show little motility. Stalked ciliates are inactive; some may have released from their stalks. Nematodes may be observed.	Low dissolved oxygen concentration in aeration tank and/or excessive retention in secondary clarifier.
Over - Aerated	Flocs are ragged and torn looking; lots of straggler floc and colloidal particles may be seen. Sludge may settle well. Gas bubbles may immediately be seen trapped in floc.	Flocs are ragged with lots of colloidal material between them. Filaments are not protruded. Increased protozoa activity. Nematodes are present.	Dissolved oxygen concentration is too high in the aeration tank. May also be due to grease or oil that prevents gas bubble release from the floc.
Dispersed	Usually brown or grey but may be white or black in colour; may or may not settle. Supernatant is turbid with floc.	High predominance of flagellates; amoeba may be observed. Flocs are irregular in shape. Filaments may be extended.	Indicative of start up condition as a result of washout.

Table 1 - continued:

SLUDGE TYPE	SV30 OBSERVATIONS	MICROSCOPE OBSERVATIONS	TROUBLESHOOTING
Deflocculated	Flocs appear to be torn apart. Sludge is unlikely to settle.	Two different observations:	
		1. Filaments are extended Protozoa are very active but beginning to shift toward lower trophic levels. Stalked ciliates may be released from their stalk if the dissolved oxygen concentration is low.	Organic shock load or too much wasted sludge.
		2. Protozoa are inactive; some appear to be distorted or exploded. Floc may have white perimeter. Many particles will be seen between flocs.	Toxic shock.
Rising Sludge	Settles well initially, but quickly rises to surface. Musty odour. Gas bubbles can be observed in flocs.	Flocs appear normal but are of smaller size with colloidal particles between them. Filaments are not extended. Marked shift towards stalked and colonial stalked ciliates. Nematodes may be present.	Indicative of old sludge in warm weather. Gas bubbles are due to denitrification. May indicate low dissolved oxygen concentration if nematodes are present.
Floating	SV30 observations are normal but sludge is noted floating on the clarifier. Sludge is odouriferous, black on one side, and brown on the other.	Microscopic observations may be normal for the process but nematodes and/or Suctoria may be observed.	Mechanical problem with sludge removal from the clarifier. Sludge either hangs up on the clarifier walls or channels in the sludge hopper.
Bulking	Sludge is light brown in colour and settles poorly with little or no compaction at the bottom of the cylinder. May have a sweet or fruit-like odour. Supernatant is clear but there is very little of it. The SVI is 200+.	Two different observations:	
		1. A flocculated sludge with a ragged edge and large surface area. Filaments may protrude slightly. Protozoa are shifting toward lower trophic levels.	Bulking due to bound water as a result of a high F/M ratio.
		2. Filaments are profuse and bridge between flocs. Protozoa may appear to be normal. Nematodes may be observed.	Filamentous bulking due to a variety of factors including: high F/M ratio; low dissolved oxygen levels; or nutrient imbalance.

SUMMARY

A troubleshooting list, which compares the visual observations made using the SV30 Test with microscope observations, for various types of activated sludge processes and sludge types, has been presented. This list can be used for both operator training and for routine operation.

The authors do not contend that the microscope should, or even can, replace all other types of testing of the activated sludge process. However, it is contended that the microscope is a useful tool for troubleshooting the process, and that the potential for many types of operational problems can be reduced through taking an appropriate control action based on the results of regular microscope observations of the activated sludge.

REFERENCES

Buswell A. M. and H. L. Long, 1923, Microbiology and theory of activated sludge, J. Amer. Wat. Wks. Ass., 10, pp. 309-321.

Butterfield, C. T., 1935, Studies of sewage purification II. A zooglea-forming bacterium isolated from activated sludge, Pub. Health Rpts., 50, 20, 671.

Cramer, Robert, 1931, The role of protozoa in activated sludge, Ind. Eng. Chem., March issue, pp. 309-313.

Curds, C. R., and A. Cockburn, 1970a, Protozoa in biological sewage treatment processes - I. A survey of the protozoan fauna of British percolating filters and activated sludge plants, Wat. Res., 4, pp. 225-236.

Curds, C. R., and A. Cockburn, 1970b, Protozoa in biological sewage treatment processes - II. Protozoa as indicators in the activated sludge process, Wat. Res., 4, pp. 237-249.

Environment Canada, 1985, WPCO Newsletter, Volume 24.

Environment Canada, 1986, WPCO Newsletter, Volume 26.

Heukelekian, H., 1931, Partial and complete sterilization of activated sludge and the effect on purification, Sew. Wks. J., 3, 369.

McKinney, Ross E., 1956, Biological flocculation, Biological Treatment of Sewage and Industrial Wastes, Vol. 1, Aerobic Oxidation, Reinhold Publishing Co., pp. 88-100.

McKinney, Ross E., 1968, Microbiological relationships in biological wastewater treatment systems, The Structure and Function of Fresh Water Microbiological Communities Symposium, Virginia Polytechnical Institute and State University, Monograph #3, pp. 165-179.

McKinney, Ross E. and Andrew Gram, 1956, Protozoa and activated sludge, Sew. Ind. Wastes, 28, 10, pp. 1219-1230.

Pillai, S. C. and V. Subrahmanyan, 1942, The role of protozoa in the activated sludge process, Nature, 150, 525.

Pillai, S. C. and V. Subrahmanyan, 1944, The influence of protozoa in the purification of sewage, Nature, 154, 179.

Sudo, R and S. Aiba, 1984, Role and function of protozoa in the biological treatment of polluted waters, Advances in Biochemical Engineering/Biotechnology, A. Fiechter ed.

INSITUFORM - THE PIPE-WITHIN-A-PIPE

by

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The Insituform process is a patented system of relining underground pipeworks developed in the U.K. in the late 1960s. The first Insitupipe was installed in a 3,000 mm sanitary sewer for the Thames Water Authority in London, England in 1971. Tests on this Insitutube in 1983 revealed no evidence of degradation or deterioration. It was recommended that no further testing was required for another 10 years and, based on these test results, the life expectancy of the Insitupipe was determined to be well over a minimum of 50 years. Since this initial inversion, thousands of kilometres of pipe in over 30 countries around the world have been relined using the Insituform process.

MATERIALS

The Insituform process utilizes a polyester needle-punched felt tube which is lined on one side with a polyurethane or poly-vinyl-chloride coating. The felt tube serves as a carrying medium for a thermosetting resin which, when cured, forms a hard impermeable liner; literally creating a new pipe within the old. The polyurethane coating serves to contain the resin during inversion and resists abrasion once normal flow is restored.

DESIGN CONSIDERATIONS

Each Insitutube is designed to meet site specific conditions. The felt tube is manufactured to the exact inside diameter and length of the existing pipeline which results in a minimal loss of cross-sectional diameter. In fact, with a Manning's "n" factor of 0.009, the Insitutube will often increase the pipe's flow capacity. In addition, since the Insitutube is a continuous pipe, there are no joints which will catch debris. This results in lower maintenance costs. Finally, the thickness of the Insitutube is designed to account for;

1) the degree of loss of structural integrity in the existing pipe,
2) hydro static load,
3) the static soil load,
4) surface surcharge load, and
5) internal pressures if a pressure pipe is being relined.

(see Appendix A)

The resin used is also designed to meet site specific conditions. Site conditions, in this case, refer predominantly to the type of liquid in the pipe. The resin mix is designed to insure that the Insitutube is not broken down by this liquid. Resins typically used include polyester, epoxy and vinylester.

METHODS OF INSTALLATION

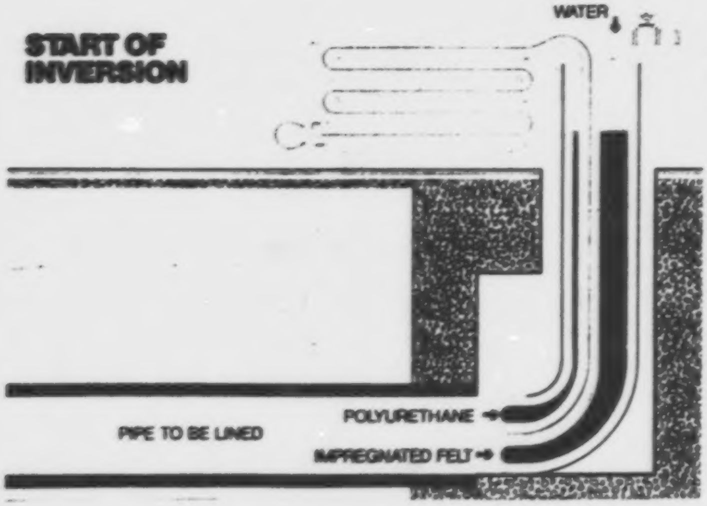
There are two methods of installing the Insitupipe. They are:

1) Bottom Inversion
2) Top Inversion

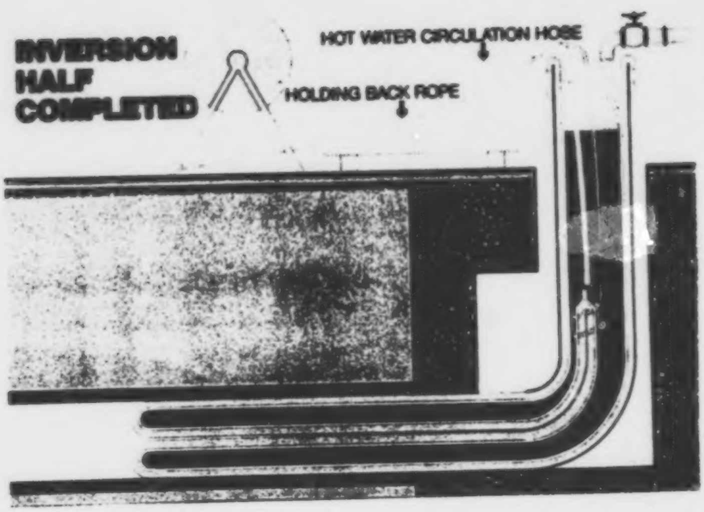
These apply after the preparation procedure has been completed. The pipe to be lined would first be cleared of all loose debris, heavy scale and projecting laterals. Any point repairs such as in areas of total pipe collapse must also be completed prior to the inversion of the Insitupipe.

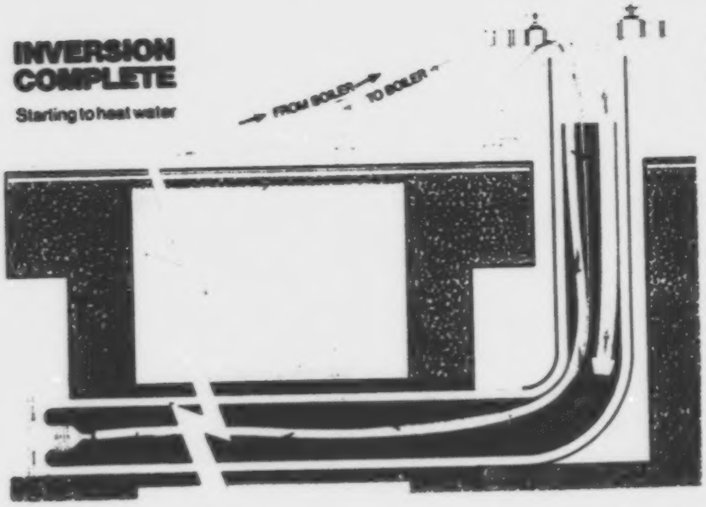
The Bottom Inversion involves the use of a steel inversion trunk with a 90° elbow at the bottom end. The elbow is usually lowered down the upstream manhole and serves to guide the Insitutube into the existing pipe. The outer coated felt is then turned over the lining to a length slightly greater than the inversion pipe. The remaining felt is turned back one metre. After the lining is fed down the inversion tube and emerges at the bottom, the ends are clamped to the top of the inversion elbow. The Top Inversion is very similar to Bottom Inversion. The difference is that the Insitutube is clamped to an inversion ring at the top of the manhole and inverted down a felt tube into the existing pipe. Thus a lining one meter in diameter can be fed into a 457mm manhole.

Water is then pumped into the space between the lining and the turned-back felt to a pre-determined head. The resulting pressure causes the Insitutube to turn inside out and travel through the existing pipe. [1]


START OF INVERSION

In small diameter pipes, the head of water is often insufficient to overcome the friction caused by the Insitutube inverting on itself. In this instance, the Pull Tape method is used in conjunction with one of the aforementioned methods. A rope is laid inside the Insitutube during its manufacture. During the Inversion, this rope is pulled in conjunction with the stepping up of the head to aid the movement of the Insitutube through the pipe.





When the Insitutube is halfway through the pipe, a hold back rope and lay flat hose are attached to the grailing end. The hold back rope is used to control the speed of the Inversion. (2).

The lay flat hose is then connected to a 7,000,000 BTU boiler. The water in the Insitutube is circulated through the boiler and raised to a temperature of 80°C. This causes an exotherm to begin in the resin and, within 1/2 hour, the initial cure is completed. The resin is then post cured during the time it takes the water to cool down to 30°C. The result is a cast in place, pipe-within-a-pipe. (3) Once the water has cooled down, the Insitutube is pierced in the down stream manhole and the water is allowed to drain out. Great care is taken to regulate this flow so that an excessive internal vacuum does not form and cause the Insitutube to collapse on itself.

The Insitutube ends are then trimmed at the manholes and, if necessary, they are grouted to the manholes to form a complete seal.

The laterals are reconnected from within the pipe. There is no need for excavation. In man-entry pipes, 2 men travel through the pipe, locate the laterals by the dimples in the liner, and cut them out with a diamond-tipped saw.

In no-man-entry pipes, the laterals are reinstated with a remote control "Insitucutter". The Insitucutter is placed in the pipe in conjunction with a CCTV camera. The laterals are again located by the characteristic dimples and reinstated by the "Insitucutter".

The entire lining process from site set-up to the reinstatement of the laterals, is typically completed within 24 hours on a 100 metre run. Since most of the work is performed at night, the inconvenience to traffic and residents is kept to a minimum.

The Insituform process has been used in pipes ranging in:

1. size from 100 mm to 2743 mm and larger.
2. shape from round clay to egg-shaped brick.
3. length from 20 m to 500 m.
4. use from municipal to industrial.
5. location from the Soviet Union to the USA to South Africa.

Typical savings in cost over excavation/replacement range from 10-40% and in some instances, such as inverted siphons, Insituform is the only practical solution.

SUMMARY

In summary, Insituform is an economically viable alternative to excavation/replacement and other forms of rehabilitation. Any shape and size will do. Inconvenience to traffic and residences is kept to a minimum. No excavation is necessary. Problems of infiltration/exfiltration, root infiltration and loss of structural integrity can be solved. The Insitutube is a cast-in-place pressure pipe. Often the flow capacity can be increased.

CONCLUSION

In conclusion, Insituform is the only alternative.

APPENDIX A

INSITUFORM TEST RESULTS

Abrasion Test - Polyurethane film

The abrasion resistance of a sample of polyurethane material bonded to a resin block was compared to asbestos-cement board by use of a Taber Abrader per test method ASTM D-1044. A weight of 500 grams was applied to each of the H-22 Calibrase wheels.

After 10,000 cycles, the urethane coating showed less than 1 mils depth of wear. The abrasion removed the surface gloss.

Abrasion Test - Resin Material

The abrasion resistance of a sample of resin material was compared to concrete using a Taber Abrader and Calibrase H-18 wheels. The resin was tested with the surface dry and again with the surface wet.

After 2000 cycles, the resin in the dry state showed 0.86 mm of wear depth and in the wet state showed 0.61 mm of wear depth.

The concrete experienced a wear depth of 0.64 mm after 2000 cycles.

Freeze-heat Cycle Test - Resin Material

Three 100 mm square samples of Insituform liner wall material were subjected to 5 freeze-heat cycles. Each cycle consisted of exposure for 18 hours at -4^{0}C followed by an immediate transfer to an oven maintained at 82^{0}C. At the completion of each cycle, the sample was cooled to ambient temperature and examined under 7X magnification and measured for dimensional stability. Average % change in all dimensions was 0.9%. No apparent signs of cracking, crazing or blistering were noted. Dimensional changes stabilized at the end of the third cycle.

Moisture Transmission Test - Resin Material

A 760 mm column of water pressure was applied over the area of a 5 mm inside diameter pipe on the urethane side of a flat sample of wall material. The samples were visually inspected daily for any evidence of moisture transmission.

No apparent water seepage was noted after 5 days of this exposure.

Joint Deflection Test - Insituform Lined Pipe

Two lengths of 200 mm I.D. pipe containing joints were cut from a pipeline after lining with Insituform liner. Each pipe was placed in a pipe deflection rig between supports 1.5 m apart.

The pipes were positioned so that the joints were at mid-span.

The first pipe failed at a displacement of 35 mm, which is equivalent to an angular deflection of 5 28' . The approximate load on the pipe at failure was 23.6 kN.

The second pipe failed at a displacement of 30 mm, which is equivalent to an angular deflection 4 38' . The approximate load on the pipe at failure was 28.1 kN.

Both pipes had been lined with a single layer (3mm) liner.

Pipe Shear Test - Insituform Lined Pipe

Three 1.52 mm lengths of 200 mm I.D. steel pipe were lined with a single layer of Insituform liner. Each pipe had a 3mm wide circumferential cut made half way along its length. The cut removed the metal completely, but did not damage the lining material. The pipes were clamped in a deflection rig and forced apart in such a manner that the centre lines of the two halves remained parallel and the gap remained at 3mm. The applied load was recorded throughout the tests and the deflection was recorded only on the tests of the second and third pipes.

The first pipe failed with an applied load of 45.7kN.

The second pipe failed with an applied load of 38.6kN at a deflection of 9mm, while the third pipe failed at 36.8kN and deflection of 10mm.

Both the deflection tests and the shear tests show that the liner is not as strong as metal pipe, but is more flexible. In the event of earth movement or settlement causing a pipe to fracture, the liner will absorb some deflection. The liner will fracture only if the soil load remains sufficiently high to strain it to its breaking point.

Fatigue Test - Resin Material

An Insituform liner can be subjected to fatigue from two sources: traffic vibration and pressure cycling. Traffic vibration is normally low stress at a high frequency; about 20HZ. Pressure cycling is low frequency, but it may be high stress.

A 3mm thick plaque of cured resin was obtained. Fatigue specimens were cut from it and tested. Testing was carried out in accordance with British Standards. The loading cycles were made to follow a sine wave form at a frequency of 10HZ. The loading was in tension.

The first specimen was load cycled over the range 3.3-9.9N/mm and it survived 10 cycles without any evidence of failure. This is equivalent to a 3 mm thick lining in a 200 mm diameter pipe being pressure cycled once a day for 2740 years. The second and third specimens were cycled between 3.3 and 17.2N/mm. One fractured after 50,000 cycles and the other after 70,000 cycles. This suggests that a 3mm thick liner in a 200 mm diameter pipe would fail after 140 years of daily pressure cycling between 0.10 and 0.54 N/mm.

The final specimen tested was cycled between 1N/mm and 20N/mm^2. It failed after 2,390 cycles.

It is interesting to note that failure on all of the fatigue specimens was sudden. Because the material is translucent, a light can be shown through the specimen under test to look for propagation cracks. This was done and no cracks were seen before the point of failure. There were some small wrinkles in one surface on the specimens, but fatigue failures did not occur at points which might have been thought to have been stress risers.

The effect of changing the cycling frequency was not investigated; however, the tests carried out suggest that the fatigue properties of the material are entirely adequate.

Insituform Enhancement Factor K

Very comprehensive laboratory testing of Insituform liners contained in rigid pipes has been carried out to establish the buckling strength of a 'supported' Insituform liner. The minimum value of K obtained was 7 with the majority of pipes yielding K values of as much as 20. For further details, please refer to Insituform.

COMPUTERIZED MAINTENANCE MANAGEMENT SYSTEM

by

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INTRODUCTION

Nationwide, billions of dollars have been spent on construction and equipment for upgrading or expansion of municipal and private water systems. In addition, most water treatment plants will spend even more for operation and maintenance over the lifetime of the facility than the original capital construction cost.

In the early 1970s, computerized management systems developed to extend equipment life, minimize equipment breakdowns, and reduce overall maintenance costs. However, because management systems require storage, retrieval, and manipulation of a large quantity of data concerning equipment items, maintenance tasks and materials, and maintenance costs, computer systems were too expensive for many municipalities and private operators.

The development of an effective manual maintenance management system required a large commitment of clerical time to paperwork. Also, use of a manual maintenance management system did not always guarantee that maintenance tasks were accomplished or that the maintenance costs were reported accurately. Periodic management reports on maintenance were also lacking. Faced with these constraints, it is not surprising that many water treatment plants have not developed an effective maintenance management system.

Recent advances in technology have lowered the cost of computers significantly so that the "micro" or "personal" computer is now affordable by even small municipalities. This paper describes a computerized maintenance management system that assists the water treatment plant maintenance manager or operator in scheduling required preventive maintenance tasks, monitoring corrective maintenance frequency and cost, accessing information about specific equipment, monitoring maintenance employee utilization, and preparing maintenance management reports.

GOALS OF A MAINTENANCE MANAGEMENT SYSTEM

In many water systems the predominant form of maintenance is corrective or repair maintenance that is performed when a piece of equipment breaks down. If periodic maintenance is performed, it usually only includes a set of generic guidelines, such as "changing the lubricant in gear drives every six months". Many programs lack a complete equipment list, a list of proper procedures and required materials, and a specific time requirement for servicing each maintenance item. A major problem limiting the development of adequate maintenance programs is that management does not know the true cost of maintenance. Many of these costs are buried in bookkeeping overheads or a recordkeeping system has not been established to monitor the costs.

An effective maintenance management system is designed to achieve the following goals:

- Provide periodic, timely, standardized and complete equipment maintenance.
- Prevent excessive maintenance, such as overgreasing of bearings.
- Increase plant reliability by preventing or providing early detection of equipment malfunction.
- Improve the efficiency of equipment operation.
- Improve plant safety by reducing unexpected breakdowns and by providing safety precautions with maintenance procedures.
- Reduce overall maintenance costs.
- Provide management with a complete record system concerning equipment history, maintenance costs and work loads.

In order for a maintenance management system to be successfully implemented and maintained, it should be relatively simple to operate with maximum output for minimum input. The use of a computer for storing and logging maintenance information achieves this objective.

ESSENTIAL COMPONENTS OF A COMPLETE MAINTENANCE MANAGEMENT SYSTEM

In developing a maintenance management system, a number of maintenance components were determined to be necessary. These are:

- Complete equipment records and maintenance history.
- Preventive maintenance scheduling.
- Corrective maintenance cost reporting.
- Standardized preventive maintenance procedures.
- Management reports on maintenance costs, overdue tasks, employee utilization, etc.

The most important component of an effective maintenance management system is maintenance personnel and organization. This includes providing adequate staffing, developing job descriptions and an organizational chart and providing maintenance training programs and periodic staff meetings.

Job descriptions are important to develop because they are used in assessing the skill level required to perform the various tasks in a maintenance program. Depending on the size of the water system, complexity of equipment, and the number of maintenance staff, various skill levels may include operator I and II, mechanic, electrician, etc. In very small plants a single individual, such as an operator, generally performs all maintenance tasks and, therefore, should have overall capabilities. Specialized system maintenance may require the use of outside contractors.

The maintenance management system must be thought of as a "tool" to be used by management and not as a substitute for the technical capabilities of the personnel who direct and perform system maintenance.

COMPUTERIZED MAINTENANCE MANAGEMENT SYSTEM

The COMPs (Computerized Operation and Maintenance Programs) Maintenance Management System was developed for use on the microcomputer. The system is intended to be used in treatment plants and pump stations with small- to medium-sized data bases. It can also easily be used in distribution or for vehicle maintenance.

The COMPs Maintenance Management System is made up of four modules, each pertaining to a specific area of maintenance management. These are:

- o Preventive Maintenance
- o Equipment Information
- o PM Procedures
- o Work Orders

Each of these modules can be used independently of the others, but using all four modules together will enable the plant staff to institute a complete and efficient maintenance management system.

The program is divided into three major sections to make it easier for the user. These areas are reports, updates and daily entries. (See the program flow chart in Figure 1).

The Reports section is divided into the four modules described above. Each report can be viewed on the screen or sent to the printer for a hard copy. There are 16 reports which give the operator the ability to obtain information in many different ways on any piece of equipment in the water system.

The Updates section is also divided into the four modules described above. It is used to add system items to the program, change information about the equipment items (edit) and delete equipment items for the program.

The third and final section of the program is the Daily Entries section. This section is for specific data entry for two of the modules: Preventive Maintenance and Work Orders. The PM Task Completion Update section provides an easy method of entering a new "Last PM Date" for the PM procedures as they are performed by the maintenance staff. The Work Order Data Entry section provides an easy method for entering specific data on the work orders performed at the plant. This information is stored by the program so it can be accessed by way of the Work Orders Report section as desired at a later date.

MENU OPERATION

The program is accessed by the operators from a series of menus. This method was used to enable an operator with little or no knowledge of computer operation to obtain output (reports) by selecting options from a sequence of menus displayed on the screen. The program can easily be operated by using only the index finger.

All program execution begins at the main menu, which is organized as shown in Figure 1. After selecting the desired option from the main menu and entering a letter A through J, the desired submenu appears. The user then selects the desired option by entering the number of that option. A third menu then appears which allows the user to input the specific data required to perform the desired task. Figure 2 shows the sequence of menus to obtain a PM Due Report.

DATA BASE DEVELOPMENT AND INPUT

The maintenance management system is only as good as the information in the data base. The identification and gathering of this information should be well planned so the final product is a usable one. The following discusses the sequence of tasks required to establish a usable system.

Equipment Identification for Maintenance

The first task in establishing a maintenance program is to determine which of the system equipment items will be input into the data base. The more extensive the list of equipment, the larger the data base becomes. The size of the data base will determine which computer system to purchase and the amount of disk storage required.

The best way to determine which items are to be entered into the program is to determine priorities and select equipment accordingly. The priorities for determining the order of equipment selection are:

- Priority 1: All equipment that must be functional to ensure the safety of system personnel and the public.
- Priority 2: All supply equipment critical to meeting anticipated demands.
- Priority 3: All standby supply equipment and that equipment not critical to meeting anticipated demands.
- Priority 4: All system equipment and structures providing for the support, comfort and aesthetics or system personnel and the public.

These four priorities should enable the plant staff to group the plant's equipment to make the selection of equipment items more straightforward.

Plant Staffing

Plant staffing is a critical area in determining the number of equipment items to include in the program. If there is not enough plant staff available to perform the PM Procedures when they become due, the program will fail. The tasks must be completed in a reasonable amount of time after they become due.

Two strategies can be taken when presented with the problem of understaffing. The first is to prioritize the equipment on how critical each piece is to the

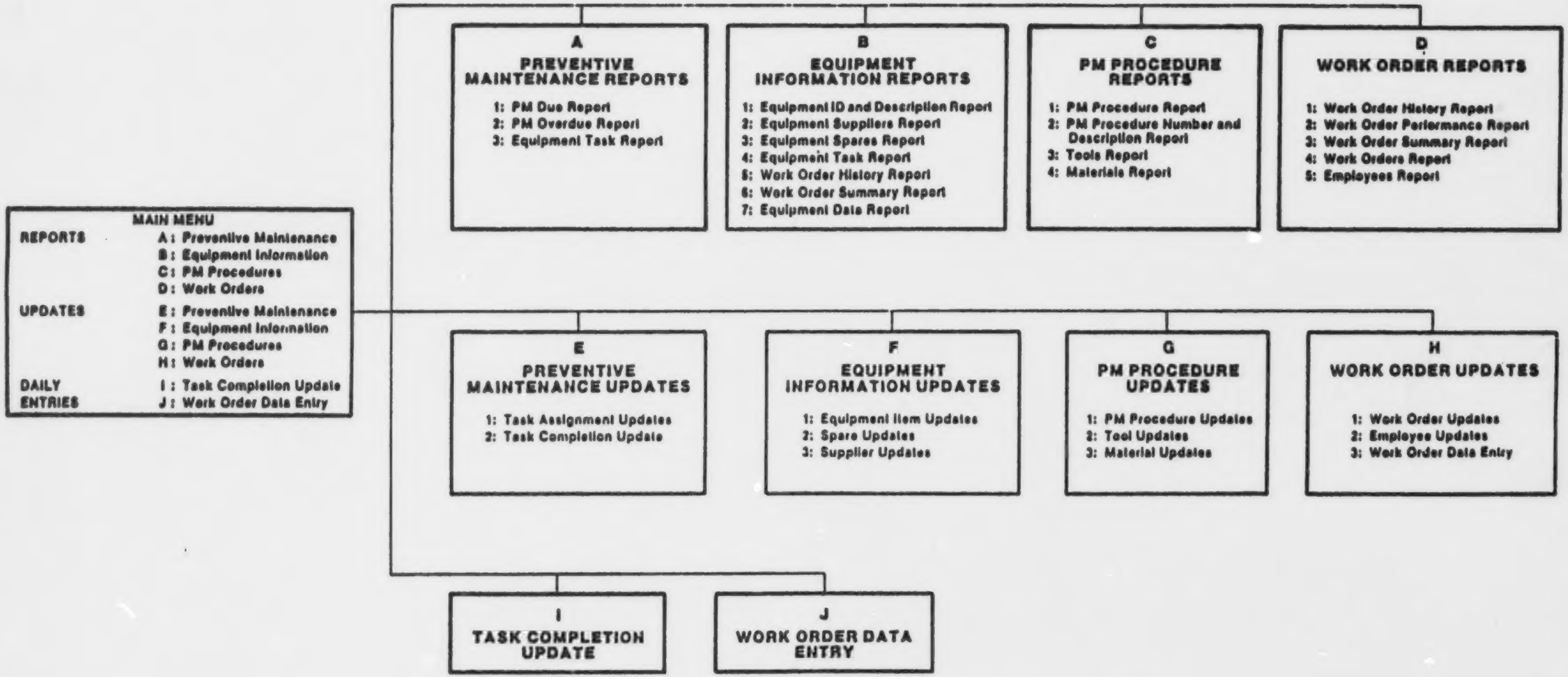
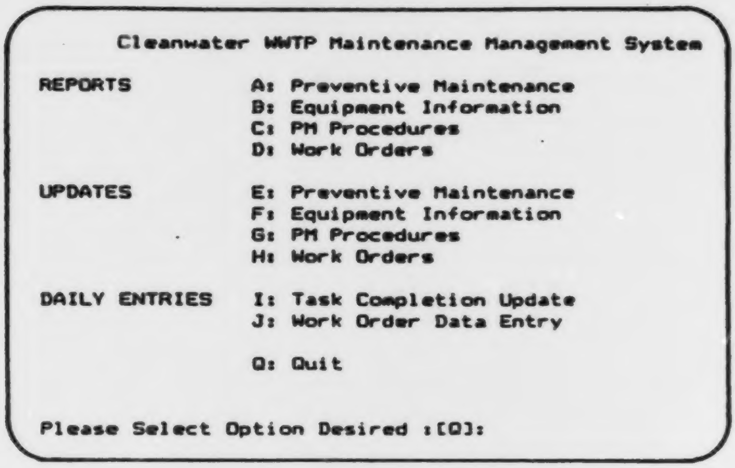

MAIN MENU
REPORTS
A: Preventive Maintenance
B: Equipment Information
C: PM Procedures
D: Work Orders
UPDATES
E: Preventive Maintenance
F: Equipment Information
G: PM Procedures
H: Work Orders
DAILY ENTRIES
I: Task Completion Update
J: Work Order Data Entry
A
PREVENTIVE MAINTENANCE REPORTS
1: PM Due Report
2: PM Overdue Report
3: Equipment Task Report
B
EQUIPMENT INFORMATION REPORTS
1: Equipment ID and Description Report
2: Equipment Suppliers Report
3: Equipment Spares Report
4: Equipment Task Report
5: Work Order History Report
6: Work Order Summary Report
7: Equipment Data Report
C
PM PROCEDURE REPORTS
1: PM Procedure Report
2: PM Procedure Number and Description Report
3: Tools Report
4: Materials Report
D
WORK ORDER REPORTS
1: Work Order History Report
2: Work Order Performance Report
3: Work Order Summary Report
4: Work Orders Report
5: Employees Report
E
PREVENTIVE MAINTENANCE UPDATES
1: Task Assignment Updates
2: Task Completion Update
F
EQUIPMENT INFORMATION UPDATES
1: Equipment Item Updates
2: Spare Updates
3: Supplier Updates
G
PM PROCEDURE UPDATES
1: PM Procedure Updates
2: Tool Updates
3: Material Updates
H
WORK ORDER UPDATES
1: Work Order Updates
2: Employee Updates
3: Work Order Data Entry
I
TASK COMPLETION UPDATE
J
WORK ORDER DATA ENTRY

FIGURE 1
Maintenance Management
System Program Flow Chart

MENU 1



```
     Cleanwater WWTP Maintenance Management System

REPORTS          A: Preventive Maintenance
                 B: Equipment Information
                 C: PM Procedures
                 D: Work Orders

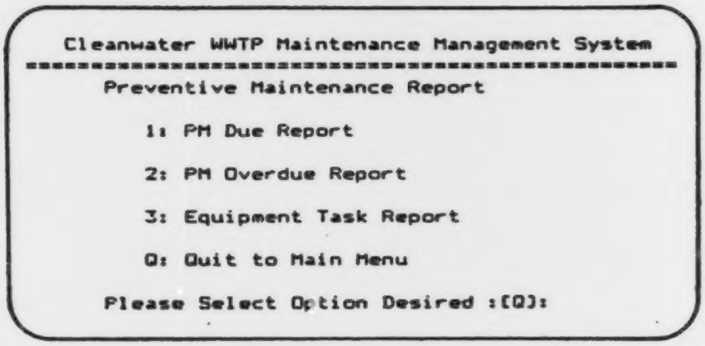
UPDATES          E: Preventive Maintenance
                 F: Equipment Information
                 G: PM Procedures
                 H: Work Orders

DAILY ENTRIES    I: Task Completion Update
                 J: Work Order Data Entry

                 Q: Quit


Please Select Option Desired :[Q]:
```

MENU 2



```
  Cleanwater WWTP Maintenance Management System
=================================================
       Preventive Maintenance Report

          1: PM Due Report

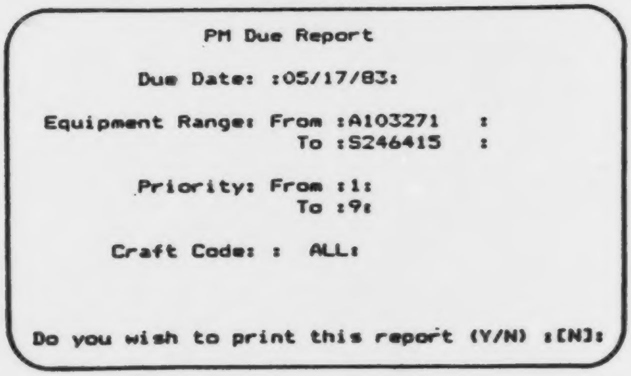
          2: PM Overdue Report

          3: Equipment Task Report

          Q: Quit to Main Menu

       Please Select Option Desired :[Q]:
```

MENU 3



```
              PM Due Report

         Due Date: :05/17/83:

 Equipment Range: From :A103271   :
                    To :S246415   :

         Priority: From :1:
                     To :9:

       Craft Code: :  ALL:


Do you wish to print this report (Y/N) :[N]:
```

FIGURE 2
MENU SEQUENCE FOR
PM DUE REPORT

plant's process. Include only the equipment items that can be properly maintained by the available maintenance staff. The other equipment items will be at the mercy of Mother Nature.

The second strategy is to determine which pieces of equipment are critical to the system's performance and enter them into the program, assigning their preventive maintenance a priority 1. The preventive maintenance for other equipment items can be given lower priorities according to how critical they are to the system's performance. This allows a separate report (priority 1 report) to be printed for the critical items. The other items can still be maintained as time and staffing allow. This strategy also provides the documentation necessary for the plant staff to request an increase in maintenance staff.

Equipment Numbering System

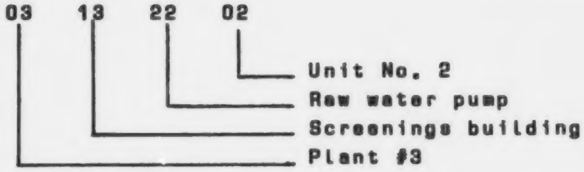
The equipment numbering system is the heart of the maintenance management system. The numbering system should give the operator the ability to easily identify individual items as well as categorize types and location of equipment.

The COMPs Maintenance Management System allows a 10-digit equipment number. This number can be alphabetic, numeric or a combination of the two. This flexibility enables a plant to use its existing numbering system if it has one.

The following is an example of a numbering system that could be used in a water system. The equipment ID is structured to give necessary information about equipment type and function. The equipment ID is broken down in the following way:

First two digits	Area
Second two digits	Building (or major plant area)
Third two digits	Function
Fourth two digits	Number of unit

For example, the equipment ID 03132202 can be broken down as follows:



Equipment Information Collection and Input

A series of Data Input and Collection Forms has been developed to aid the plant staff in the task of collecting and inputting the large amount of information required to set up the maintenance management system. The program's data input screens in the update section match these forms. This allows someone with little knowledge of the program to perform the tedious task of entering all of the information into the data base, thus enabling the plant staff to put their efforts into organizing the information and filling out the forms. Appendix A contains some samples of these input forms.

REPORT GENERATION

The program out put is a series of predefined reports. These reports can be either printed out as hard copy on a printer or they can be displayed on the screen. Each report is accessed through a series of menus starting with Options A through D of the main menu.

Preventive Maintenance Reports

The COMPs Maintenance Management System Preventive Maintenance module prints three reports of use to the maintenance worker and manager. These reports provide the information needed to schedule preventive maintenance tasks and to keep track of preventive maintenance tasks that are overdue.

PM Due Report. The PM Due Report is used to obtain information about the scheduled PM tasks that need to be performed on the system's equipment. A report can be printed at any time for any due date desired. The best use of this report is to print it out weekly. This can be done by printing the report each Friday with a due date of the following Friday. This allows the scheduling of staff for the following week. At the end of the week, prior to the printing of the next week's report, the new, last PM date should be entered for each PM task that was performed. Those tasks that were not performed will print out again on the following week's report.

Each PM procedure is assigned a priority and a craft code when the program is set up. This gives the user the ability to obtain PM Due Reports for tasks with a specific priority and/or craft code.

Another feature on this report is that all procedures that are more than 10 percent overdue are flagged with three asterisks. This notifies the maintenance worker of overdue tasks.

PM Overdue Report. The PM Overdue Report gives the maintenance manager a tool with which to monitor the ability of the maintenance crew to keep up with the scheduled PM. This report is essentially the same as the PM Due Report. The difference is that it will only include the PM procedures that are more than 10 percent overdue.

For instance, a task that should be done every 100 days will not be printed out if the difference between the due date and the last PM date is 109 days. If it is 112 days, the task is 12 percent overdue and will be printed out. The percent overdue is also printed with each procedure on this report to give the manager an idea of how serious the problem is. This report can also be limited by priority and/or craft code.

Equipment Task Report. The Equipment Task Report lists the PM procedures that have been designated for each piece of equipment.

Equipment Information Reports

The Equipment Information module contains specific information on each piece of equipment in the program. This includes such information as the supplier, recommended spare parts, serial number, startup date, catalog number and nameplate data. This module provides a place to store a large amount of "equipment information" that is easily misplaced or lost in the water system.

Equipment Data Report. The report provides a summary of all the information input into the program on a specific equipment item. This is an extremely useful tool that makes it an easy and fast task to find information that would normally be in more than one place if it could be found at all. This information includes:

- Equipment number and description
- Manufacturer
- Model and serial numbers
- Catalog and drawing numbers
- Supplier, with address and telephone number
- Initial equipment cost and startup date
- Nameplate data
- Recommended spare parts list

PM Procedure Reports

The PM Procedures module is where all of the tasks performed as preventive maintenance on the plant equipment are defined.

PM Procedures Report. This report shows the tools required, materials required and special instructions for performing each PM procedure. It gives system staff a place to document helpful hints, lubricant types, safety procedures and useful tools that will help in performing the specific PM procedures. The user can easily call up this report when assigned the PM procedure and obtain information that will make the job easier and safer. When new experiences need to be recorded, they can be added through the PM Procedure Updates section of the program.

Work Order Reports

The COMPs Maintenance Management System defines all maintenance tasks in the system as one of two types: planned or scheduled maintenance and breakdown or corrective maintenance. All breakdown or corrective maintenance is defined as Work Orders in the COMPs Maintenance Management System.

The program contains a module that allows the system staff to enter specific information from work orders that have been performed. This information is then available to the reports section for reports that give the staff information that can aid in identifying problem equipment and costs associated with work orders.

Work Order History Report. This report lists all of the work orders performed for a specified equipment item. Information included with each work order is the work order ID, description of work performed, manhours worked and material cost.

This is a very easy way to search through years of work orders and quickly come up with the ones performed on specific equipment. All of the original work orders should still be filed in case more specific information is needed. These can be easily found if filed in logical order or work order ID.

__Work__ __Order__ __Summary__ __Report__. This report lists the total number of work orders performed on a piece of equipment over the designated data range. Also included is the total labour cost, material cost, initial equipment cost and total maintenance cost/initial equipment cost (TC/IC) factor.

This report is very useful in identifying problem or high maintenance equipment. The TC/IC Factor shows the relative cost of work order maintenance to the equipment's value. The report will allow easy justification for replacement of high maintenance items.

__Work__ __Order__ __Performance__ __Report__. This report compares the planned time required to perform the work order with the actual time taken. It provides the maintenance manager with a tool to determine if the time allotted to perform the work orders is accurate. Estimating and planning the time to perform work orders is very important if the maintenance manager wants to be sure he is able to keep up with the required work load.

SUMMARY

The reports described above are just the major reports printed by the program. Other reports are available which can be identified on Figure 1. Examples of some of the reports can be found in Appendix B.

PROGRAM COMPATIBLE HARDWARE

The maintenance management program was written in the CP/M operating system so it can operate on many of the microcomputers available on the market today. The microcomputers on which the system is now running include the Apple II, Apple IIe, TRS 80 Model II, IBM PC, IBM XT and the DEC Rainbow.

The minimum system requirements include:

- IBM PC=XT with 256K RAM, 360K floppy disk drive 10 MB hard disk drive
- IBM Color Graphics Card
- IBM Color Monitor
- AST SixPakPlus, 384K, S/P/CC & S/W
- Okidata 193 Printer with IBM Compatible ROM
- Printer Cable
- DOS 3.1 operating system

Note: The COMPs programs currently require 512K RAM. The components of the system shown above can vary. For instance, some people may prefer a monochrome monitor with higher resolution or a different kind of printer. Please confirm the compatibility of any variance with a COMPs representative.

The system staff should carefully choose their computer hardware so it will meet the needs of the water system. They should be certain the disk storage capacity is large enough to handle their total data base. The staff should also keep computer uses other than maintenance management in mind when selecting their system. Other uses include:

- Inventory control
- Process control
- Industrial monitoring
- Monthly monitor reporting
- Word processing

The plant staff should always keep in mind that the proper hardware is essential to operate their software, yet the software is what is required to give the desired results.

MAINTENANCE MANAGEMENT SYSTEM IMPLEMENTATION

After the decision has been made to install a computerized maintenance management system, there are a few important considerations prior to and during implementation:

- Coordination of operations and maintenance
- Acceptance by mechanics
- Implementation schedule

Coordination between the operations and maintenance departments requires that the responsibilities of each department for preventive maintenance tasks or work orders be established while the program is being developed. Meetings and written communication between the two departments must be encouraged. Just as the maintenance department must contact the operations department before removing equipment from service for maintenance, the operations department must consult the maintenance department before performing unscheduled PM tasks, such as lubrication.

In order for a maintenance management system to work, it must be accepted by the maintenance staff. This can be accomplished by obtaining their input and review during system development and when assembling equipment information, PM tasks and procedures and labour requirements.

A new maintenance management program should be installed gradually to enable maintenance personnel to become familiar with the PM program, work orders, etc. This could be accomplished by adopting the program in stages, such as booster stations. As the staff gains experience and confidence with the system, more equipment items and unit processes can be added.

One problem with staged implementation is that the staff must be informed which equipment items are currently included in the system; thus, should a piece of equipment not included break down, it will not be blamed on the maintenance management system.

In summary, a successful maintenance management system requires proper planning, an adequate budget, the support of management and staff and a reasonable testing period.

REFERENCES

CH2M HILL. Preventive Maintenance System Users Manual. Prepared by the Upper Occoguan Sewage Authority, Manassas Park, Virginia. December 1978.

Holst, A.M. and J.L. Puntenney. "Make Your Maintenance Records Work for You." Deeds & Data. Water Pollution Control Federation. March 1980.

Jordan, J.K. "Preventive Maintenance and the Minicomputer: Automation with Low Cost." Washington Suburban Sanitary Commission, presented at the 1982 Water Pollution Control Federation Conference. October 7, 1982.

Marinello, R.L. "Computerizing Maintenance Information." Plant Engineering. February 18, 1982.

Morrow, L.C. Maintenance Engineering Handbook. 2nd Ed. McGraw-Hill Book Co., New York, N.Y. 1966.

Newbrough, E.T. Effective Maintenance Management. McGraw-Hill Book Co., New York, N.Y. 1967.

Appendix A

DATA INPUT AND COLLECTION FORMS

EQUIPMENT INPUT FORM

Equip ID.: ----------: **Equip Description:** ------------------------------:
Manufacturer: ---------------:
Model No: --------------------: **Catalog ID:** ---: **Supplier ID:** ----:
Serial No: --------------------: **Drawing ID:** ---:
Initial Cost: --------:
Startup Date: --/--/--:

Type 1: Drive Unit
HP: ---: **Volts:** ---: **Amps:** ---: **Phase:** -: **RPM:** -----:
Frame: ------------: **Type:** ------------:
:--:

Type 2: Driven Unit
Capacity: --------: **RPM:** -----: **TDH:** --------: **Type:** ------------:
Impeller: ------------: **Packing:** -----------:
:--:

Type 3: Intermediate Unit
RPM In: -----: **RPM Out:** -----:
:--:

Type 4: Valve
Type: --------: **Size:** ------: **Operator:** -----------:
:--:

Type 5: General
Identification: ---------------:
:--:
:--:

RECOMMENDED SPARE PARTS

Spare ID.	Spare Description	Spare ID.	Spare Description
:----:	:------------------------------:	:----:	:------------------------------:
:----:	:------------------------------:	:----:	:------------------------------:
:----:	:------------------------------:	:----:	:------------------------------:
:----:	:------------------------------:	:----:	:------------------------------:
:----:	:------------------------------:	:----:	:------------------------------:

---

* **Note: Equipment ID's must be 10 characters or less, all ID's must have the same no. of characters.**
* **Note: Equipment description must be 30 characters or less.**

PM TASKS

Equip. ID: . . . . **:** **Equip. Description:** . . . .

Task No.	Prior.	PMP No.	Description	Interval	Last PM
——	—	——	————————	——	—/—/—
——	—	——	————————	——	—/—/—
——	—	——	————————	——	—/—/—
——	—	——	————————	——	—/—/—
——	—	——	————————	——	—/—/—
——	—	——	————————	——	—/—/—
——	—	——	————————	——	—/—/—
——	—	——	————————	——	—/—/—
——	—	——	————————	——	—/—/—
——	—	——	————————	——	—/—/—
——	—	——	————————	——	—/—/—
——	—	——	————————	——	—/—/—
——	—	——	————————	——	—/—/—
——	—	——	————————	——	—/—/—
——	—	——	————————	——	—/—/—
——	—	——	————————	——	—/—/—
——	—	——	————————	——	—/—/—
——	—	——	————————	——	—/—/—
——	—	——	————————	——	—/—/—
——	—	——	————————	——	—/—/—
——	—	——	————————	——	—/—/—
——	—	——	————————	——	—/—/—
——	—	——	————————	——	—/—/—
——	—	——	————————	——	—/—/—
——	—	——	————————	——	—/—/—
——	—	——	————————	——	—/—/—
——	—	——	————————	——	—/—/—
——	—	——	————————	——	—/—/—
——	—	——	————————	——	—/—/—

PREVENTIVE MAINTENANCE PROCEDURE (PMP) INPUT FORM

PMP No*: ---: **Description*:** --:

Craft Code: -----: **Time Required:** ----:

TOOLS

Tool ID	Tool Description	Tool ID	Tool Description
:----:	:---------------:	:----:	:---------------:
:----:	:---------------:	:----:	:---------------:
:----:	:---------------:	:----:	:---------------:
:----:	:---------------:	:----:	:---------------:
:----:	:---------------:	:----:	:---------------:

MATERIALS

Material ID	Material Description	Material ID	Material Description
:----:	:---------------:	:----:	:---------------:
:----:	:---------------:	:----:	:---------------:
:----:	:---------------:	:----:	:---------------:
:----:	:---------------:	:----:	:---------------:
:----:	:---------------:	:----:	:---------------:

SPECIAL INSTRUCTIONS:

:---:

:---:

:---:

:---:

:---:

:---:

:---:

:---:

:---:

- **Note: All PMP Nos. must be 3 characters or less, all PMP Nos. must have the same no. of characters.**
- **Note: PMP No. Descriptions must be 40 characters or less.**

PM MATERIALS LIST

Material ID:*	Description*	Material ID:*	Description*

- * Note: Material ID's must be 4 characters or less, all ID's must have the same no. of characters.
- * Note: Material description must be 15 characters or less.

Appendix B

SAMPLE REPORTS

```
                              PM DUE REPORT
              Cleanwater WWTP Maintenance Management System
                                 08/25/83                      Page    1

Equipment Range: 011111    - 134110       Priority Range: 1 -  9   Due 08/25/83
==============================================================================
Equipment         --- Equipment Description ---
  ID              PMP                                             Over    Craft
   Task No. Pri   No.  ----PM Procedure ---                       Due     Code
==============================================================================
011111            123RD St PS Pump No. 1 Motor
    001      1      012 Lube upper and lower motor bearings               Mech
011112            123RD St PS Pump No. 1 Pump
    001      1      014 Inspect, tighten, or change packing               Mech
011121            123RD St PS Pump No. 2 Motor
    001      1      012 Lube upper and lower motor bearings               Mech
011122            123RD St PS Pump No. 2 Pump
    001      1      014 Inspect, tighten, or change packing               Mech
011510            123RD St PS Generator Set
    001      1      003 Check operation of generator & switchgear         Oper
    005      1      034 Lube motor bearings                               Mech
    006      1      018 Change engine oil                                 Mech
    007      1      020 Inspect & change air filters                      Mech
012111            Monroe St PS Pump No. 1 Motor
    001      1      012 Lube upper and lower motor bearings       ***     Mech
012112            Monroe St PS Pump No. 1 Pump
    001      1      014 Inspect, tighten, or change packing       ***     Mech
012121            Monroe St PS Pump No. 2 Motor
    001      1      012 Lube upper and lower motor bearings       ***     Mech
012122            Monroe St PS Pump no. 2 Pump
    001      1      014 Inspect, tighten, or change packing       ***     Mech
012410            Monroe St PS Wet Well
    001      3      024 Flush and clean wet well                  ***     Labr
021211            Comminutor No. 1 Motor
    001      1      012 Lube upper and lower motor bearings               Mech
021214            Comminutor No.1 Controls
    002      2      026 Inspect clean & tighten connections               Elec
021221            Comminutor No. 2 Motor
    001      1      012 Lube upper and lower motor bearings               Mech
021224            Comminutor No. 2 Controls
    002      2      026 Inspect clean & tighten connections               Elec
022120            Grit Chamber No. 2 Diffusers
    001      2      008 Clean diffusers                           ***     Oper
022213            Grit Blower No. 1 Controls
    002      3      026 Inspect, clean, & tighten connections             Elec
    003      3      025 Calibrate pressure switches                       Elec
022223            Grit Blower No. 2 Controls
    002      3      026 Inspect, clean, & tighten connections             Elec
    003      3      025 Calibrate pressure switches                       Elec
022313            Grit Pump No. 1 Controls
    002      3      026 Inspect, clean, & tighten connections             Elec
    003      3      035 Check operation of programmable timer             Elec
022323            Grit Pump No. 2 Controls
    002      3      026 Inspect, clean, & tighten connections             Elec
    003      3      035 Check operation of programmable timer             Elec
022610            Headworks Valves

                         Continued on Next Page
```

```
                    PM OVERDUE REPORT
        Cleanwater WWTP Maintenance Management System
                         08/25/83                          Page   1

Equipment Range: 011111     - 134110      Priority Range: 1 -  9   Due 08/25/83
===============================================================================
Equipment          --- Equipment Description ---
   ID                PMP                                     %Over      Craft
  Task No. Pri       No.   -----PM Procedure ---              Due       Code
===============================================================================
012111             Monroe St PS Pump No. 1 Motor
      001     1      012 Lube upper and lower motor bearings       18       Mech
012112             Monroe St PS Pump No. 1 Pump
      001     1      014 Inspect, tighten, or change packing       24       Mech
012121             Monroe St PS Pump No. 2 Motor
      001     1      012 Lube upper and lower motor bearings       18       Mech
012122             Monroe St PS Pump No. 2 Pump
      001     1      014 Inspect, tighten, or change packing       24       Mech
012410             Monroe St PS Wet Well
      001     3      024 Flush and clean wet well                  33       Labr
022120             Grit Chamber No. 2 Diffusers
      001     2      008 Clean diffusers                           14       Oper
022610             Headworks Valves
      001     2      002 Work all valves                           18       Oper
041030             Secondary Clar. #3 Structure
      001     2      011 Drain & clean basin                       40       Oper
```

```
                         EQUIPMENT TASK REPORT
             Cleanwater WWTP Maintenance Management System
                              08/25/83                         Page    1

Equipment Range: 011111     - 134110           Priority Range: 1 -   9
================================================================================
Equipment      --- Equipment Description ---
  ID       Task        PMP                                          Craft
           No. Prior   No.      --- PM Procedure ---                Code
================================================================================
011111              123RD St PS Pump No. 1 Motor
           001   1     012   Lube upper and lower motor bearings    Mech
           002   2     013   Take & record vibration readings       Mech
           003   4     023   Inspect coating & paint if necessary   Labr
011112              123RD St PS Pump No. 1 Pump
           001   1     014   Inspect, tighten, or change packing    Mech
           002   3     016   Pull pump for inspection               Mech
           003   4     023   Inspect coating & paint if necessary   Labr
011113              123RD St PS Pump No. 1 Controls
           001   1     001   Check proper sequence of operation     Oper
           002   2     025   Calibrate pressure switches            Elec
           003   2     026   Inspect, clean, & tighten connections  Elec
011121              123RD St PS Pump No. 2 Motor
           001   1     012   Lube upper and lower motor bearings    Mech
           002   2     013   Take & record vibration readings       Mech
           003   4     023   Inspect coating & paint if necessary   Labr
011122              123RD St PS Pump No. 2 Pump
           001   1     014   Inspect, tighten, or change packing    Mech
           002   3     016   Pull pump for inspection               Mech
           003   4     023   Inspect coating & paint if necessary   Labr
011123              123RD St PS Pump No. 2 Controls
           001   1     001   Check proper sequence of operation     Oper
           002   2     025   Calibrate pressure switches            Elec
           003   2     026   Inspect, clean, & tighten connections  Elec
011210              123RD St PS Sump Pump
           001   1     001   Check proper sequence of operation     Oper
           002   2     007   Clean debris from sump                 Oper
011310              123RD St PS Valves
           001   2     002   Work all valves                        Oper
           002   2     014   Inspect, tighten, or change packing    Mech
011410              123RD St PS Wet Well
           001   3     024   Flush and clean wet well               Labr
011510              123RD St PS Generator Set
           001   1     003   Check operation of generator & switchgea Oper
           002   1     004   Check engine oil level                 Oper
           003   1     005   Check & record fuel level              Oper
           004   1     006   Check battery charge                   Oper
           005   1     017   Lube bearings                          Mech
           006   1     018   Change engine oil                      Mech
           007   1     020   Inspect & change air filters           Mech
           008   2     026   Inspect, clean, & tighten connections  Elec
012111              Monroe St PS Pump No. 1 Motor
           001   1     012   Lube upper and lower motor bearings    Mech
           002   2     013   Take & record vibration readings       Mech
           003   4     023   Inspect coating & paint if necessary   Labr
012112              Monroe St PS Pump No. 1 Pump

                        Continued on Next Page
```

EQUIPMENT ID AND DESCRIPTION REPORT
Cleanwater WWTP Maintenance Management System
08/25/83

Equipment Range: 011111 - 134110

Equipment ID	---Equipment Description---
011111	123RD St PS Pump No. 1 Motor
011112	123RD St PS Pump No. 1 Pump
011113	123RD St PS Pump No. 1 Controls
011121	123RD St PS Pump No. 2 Motor
011122	123RD St PS Pump No. 2 Pump
011123	123RD St PS Pump No. 2 Controls
011210	123RD St PS Sump Pump
011310	123RD St PS Valves
011410	123RD St PS Wet Well
011510	123RD St PS Generator Set
012111	Monroe St PS Pump No. 1 Motor
012112	Monroe St PS Pump No. 1 Pump
012113	Monroe St PS Pump No. 1 Controls
012121	Monroe St PS Pump No. 2 Motor
012122	Monroe St PS Pump No. 2 Pump
012123	Monroe St PS Pump No. 2 Controls
012210	Monroe St PS Sump Pump
012310	Monroe St PS Valves
012410	Monroe St PS Wet Well
012510	Monroe St PS Generator Set
021110	Parshall Flume Controls & Alarms
021211	Comminutor No. 1 Motor
021212	Comminutor No. 1 Gear Box
021213	Comminutor No. 1 Cutters
021214	Comminutor No. 1 Controls
021221	Comminutor No. 2 Motor
021222	Comminutor No. 2 Gear Box
021223	Comminutor No. 2 Cutters
021224	Comminutor No. 2 Controls
021310	Bypass Bar Screen
022110	Grit Chamber No. 1 Diffusers
022120	Grit Chamber No. 2 Diffusers
022211	Grit Blower No. 1 Motor
022212	Grit Blower No. 1 Blower
022213	Grit Blower No. 1 Controls
022221	Grit Blower No. 2 Motor
022222	Grit Blower No. 2 Blower
022223	Grit Blower No. 2 Controls
022311	Grit Pump No. 1 Motor
022312	Grit Pump No. 1 Pump
022313	Grit Pump No. 1 Controls
022321	Grit Pump No. 2 Motor
022322	Grit Pump No. 2 Pump
022323	Grit Pump No. 2 Controls
022410	Cyclone & Washer No. 1
022420	Cyclone & Washer No. 2
022510	Headworks Bldg Sump Pump
022610	Headworks Area Valves

Continued on Next Page

```
                       EQUIPMENT DATA REPORT
             Cleanwater WWTP Maintenance Management System
                              8/25/83
Equipment Range: 041210    - 041210                             Page   1
==========================================================================
Equipment ID: 041210        Description: Sec Clar #1 Drive Mechanism

Manufacturer: Envirex                       Supplier: G.T. Morris
Model No.: H-40LT                                    2020 Lost Creek Dr.
Serial No.: H-116237                                 Salem           OR
Initial Cost: 15000    Catalog ID: A12                          97288
Startup Date:03/17/78  Drawing ID: S8                      503/292-9292
--------------------------------------------------------------------------
General
 80'-0" Diameter Tow-Bro w/ Skimming : Flow (MGD) - 2.016 Max,.504 Min :
 Velocity (FPS) - 2.657 Max, .664 Min : Headloss - .839 FT Max :

--------------------------------------------------------------------------
Recommended Spare Parts

 DCH  Drive Chain                        SPR  Drive Sprocket
 WMG  Worm Gear                          WRM  Worm
 VPG  Vent Plugs (Set)                   SQG  Squeegees for Scraper Arm
 DBD  Deflector Blade                    MSL  Manifold Seals
 SSK  Surface Skimmer Flex. Blade        MSW  Torque Overload Micro Switches
```

PM PROCEDURES REPORT
Cleanwater WWTP Maintenance Management System
08/25/83

PM Procedure Range: 001 - 010

==

PMP NO.: 001 Description: Check for proper sequence of operation

Craft Code: Oper Time Required: 0.25

Tools Required
30 : Stop Watch

Materials Required
19 : None Needed

Special Instructions

Monitor operation of equipment through one complete cycle of operation : Check both manual and auto modes : Report anything that is not normal

PMP NO.: 002 Description: Work all valves

Craft Code: Oper Time Required: 0.50

Tools Required
24 : No Tools Needed

Materials Required
19 : None Needed

Special Instructions

Open or close, then return to the original position all valves in the designated area. Take care not to close a valve against a working pump. Be sure to leave all valves in the same position that they were found. Report any leaky or difficult to open valves to the supervisor on duty.

PMP NO.: 003 Description: Check operation of generator & switchgear

Craft Code: Oper Time Required: 0.50

Tools Required
24 : No Tools Needed

Materials Required
19 : None Needed

Special Instructions

Notify plant personnel that plant will be going on emergency power : Check engine oil level : Make sure generator is on AUTO : Throw main breaker : Check - generator starting and going on line - switchgear - plant equipment going back on line - generator operation : Allow generator to reach operating temperature, at least operate 1/2 hour : Notify plant personnel that plant will be going on to main power : Throw main breaker to "on" position : Check - switchgear - generator

Continued on Next Page

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                    WORK ORDER HISTORY REPORT
          Cleanwater WWTP Maintenance Management System
                            08/25/83                        Page   1
                                             Start            End
Equipment Range: 011111    - 012410          Date: 01/01/83   Date: 08/25/83
==========================================================================
Equipment Work Order  Date WO   Work Order  Description         Man    Parts
   ID        ID       Closed                                    Hours  Cost
==========================================================================
011111      A002103  01/25/83 Replaced upper motor bearing        8.0     75
011111      A002298  03/10/83 Replaced lower motor bearing       10.0     75
011112      A002159  02/12/83 Rebuilt impeller                   12.0    350
011122      A002321  03/15/83 Rebuilt impeller                    8.0    350
011510      A001976  01/10/83 Replaced time delay relay           1.0     75
011510      A002386  03/22/83 Unplugged fuel pump                 2.0      0
011510      A002391  03/25/83 Cleaned out fuel storage tank      16.0      0
012111      A002468  04/22/83 Rewound motor                       4.0    450
012112      A002117  02/06/83 Replaced shaft sleeve               8.0    115
012121      A002470  04/22/83 Rewound motor                       4.0    450
011210      A002573  06/22/83 Replaced burned out pump            3.0    125
011510      A002501  05/03/83 Replaced battery                    1.0     78
011510      A002503  05/03/83 Replaced tach                       4.0    230
```

```
                           WORK ORDER SUMMARY REPORT
                 Cleanwater WWTP Maintenance Management System
                                   08/25/83                         Page   1
                                                Start            End
Equipment Range:  011111 - 012510               Date:01/01/78    Date:08/25/83
==============================================================================
Equipment    No. of           Labor         Parts     Total   Initial  TC/IC
    ID       WO's             Cost          Cost      Cost    Cost
==============================================================================
011111        4              $  353       $  190   $  543   $  850      0.64
011112        2              $  154       $  415   $  569   $  775      0.73
011113        1              $   30       $    0   $   30   $  950      0.03
011121        1              $   45       $   12   $   57   $  850      0.07
011122        2              $  160       $  395   $  555   $  775      0.72
011123        1              $   30       $    0   $   30   $  950      0.03
011310        1              $   43       $  275   $  318   $  945      0.33
011510        6              $  186       $  647   $  833   $ 8900      0.09
012111        2              $  111       $  514   $  625   $  850      0.74
012112        3              $  145       $  227   $  372   $  775      0.48
012113        1              $   34       $   14   $   48   $  950      0.05
012121        1              $   74       $  184   $  258   $  850      0.30
012122        1              $   56       $   74   $  130   $  775      0.17
012123        2              $  123       $  786   $  909   $  950      0.96
012210        1              $   35       $  110   $  145   $  105      1.38
012510        3              $  156       $  447   $  603   $ 8900      0.07
```

```
                        WORK ORDER PERFORMANCE REPORT
                  Clearwater WWTP Maintenance Management System
                                  08/25/83                          Page    1

 Start Date: 05/01/83                         End Date: 08/25/83
=================================================================================
 Employee Name                 Employee  WO     Planned    Actual    act/pld  Comment
                                  ID     ID      Hours     Hours
=================================================================================
John D. Doe                    0131  A002602     4.0        3.5      0.87
John D. Doe                    0131  A002608    12.0       14.0      1.17
John D. Doe                    0131  A002618     8.0        6.0      0.75
John D. Doe                    0131  A002625     2.0        2.0      1.00
John D. Doe                    0131  A002626     3.0        4.0      1.33
John D. Doe                    0131  A002630    20.0       16.0      0.80
John D. Doe                    0131  A002634     8.0        6.0      0.75
John D. Doe                    0131  A002638    12.0       12.0      1.00
John D. Doe                    0131  A002655     6.0        7.5      1.25
John D. Doe                    0131  A002661     4.0        4.0      1.00
John D. Doe                    0131  A002665    10.0       11.0      1.10
John D. Doe                    0131  A002671    12.0        8.0      0.67
John D. Doe                    0131  A002673     6.0        6.0      1.00
John D. Doe                    0131  A002678     8.0       12.0      1.50
John D. Doe                    0131  A002687     8.0        8.0      1.00
John D. Doe                    0131  A002694     2.0        1.0      0.50
John D. Doe                    0131  A002699     1.0        1.0      1.00
```

LOCATING AND SIZING AIR RELEASE VALVES
IN TRANSMISSION MAINS

written by

J.F. Lescovich, Chief Engineer
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presented by

Ronald L. Jewell
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If you were to consider, for example, a typical pipeline of any diameter, and say one mile long, that has been supposedly vented of air, in most instances this same pipeline would still contain enough dissolved air to completely fill over 100 feet of the pipe. The reason is that water at standard conditions contains over 2% dissolved air by volume and on many occasions a much higher percentage. It is generally known that the solubility of air in water is governed by the line pressure and temperature. Since a column of moving water is subjected to constantly changing pressures due to the terrain changes over which it travels, it is obvious that air is constantly coming out of solution at higher points in the pipeline which are closer to the hydraulic gradient. Once out of solution, air will not as readily return into solution.

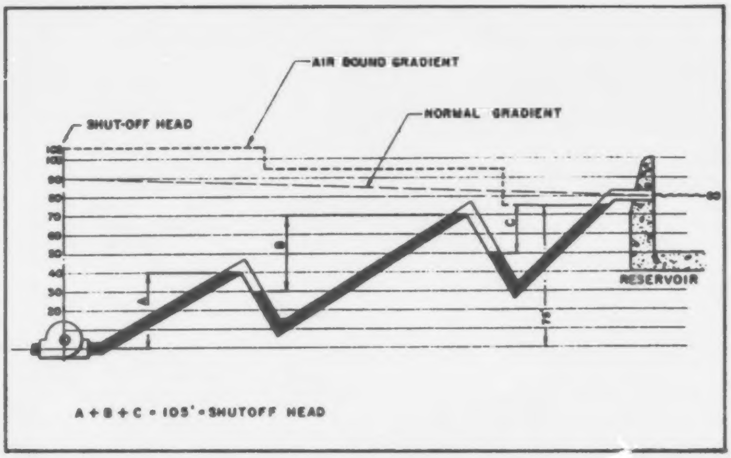
Many times designers conclude that since there are no obvious sources of air entry, there is no air in the pipe. Subsequently when the system performance fails to meet the design criteria, other factors are blamed.

The significance of air in pipelines is badly neglected. It is often felt that an air release valve located anywhere or perhaps at some principle high point in the system is adequate. This writer's experience indicates that the ventilation of a pipeline is a very involved subject. In many installations air release valves have had to be relocated or increased in size or number before a satisfactory system operation was achieved. How many times has an engineer, during design, evaluated the economics of a system and quoted design head loss figures to decimal places but was unaware of the significance of just one overlooked air pocket in the pipe. Often the pump performance is questioned or pipeline "C" coefficient is revised.

There are no doubt countless pumps running at this very moment at an increased head because air at some unsuspected location is obstructing the flow.

The presence of air pockets in a pipe can seriously reduce its carrying capacity. In fact, air binding can completely stop the flow even though the pumping pressure is more than adequate to provide flow. Figure 1 illustrates an air bound pipeline where no flow is occurring and the pump is at shutoff head.

Figure 1



Air Entry

The prevention of air from entering the pipe would go far in the elimination of the problems associated with it. Complete elimination of air is not possible however, but an understanding of how air may enter a pipe will assist in minimizing its entry.

1. Air that is in a pipeline before the line is filled and is not completely purged.

2. Well pumps on start-up can add considerable amounts of air into the pipe if no provisions are made to release the air before the check valve opens.

3. Air may be drawn in at intakes by the vortex action of the pump suction. It has been learned that air in quantities equal to 5% or 10% of the flow could be carried into a pump intake by the vortex. As well as contributing to the pipeline problems, this air has detrimental effects on the pump in the form of vibration, corrosion and inefficiency.

4. Air may be drawn into a pipeline through leaky joints at zones of negative pressure such as points which are above the hydraulic gradient.

5. Air may drawn in at pump glands.

6. Cascading flow in a partially filled line can entrap a large amount of air resulting in white water. The turbulence associated with cascading flow causes non-uniform mixture of air and water which is very difficult to ventilate.

7. Air may be entrapped by the hydraulic jump resulting from the flow through a partially opened gate.

8. Air in solution will be released at points in the pipeline where pressure is reduced as was mentioned earlier.

Air, once in the system, tends to collect at summits in the pipeline where the air pockets will increase in size. If the fluid velocity is high, the air may be swept along with the flow automatically purging the line. In most instances, the forward movement of the water will force the bubbles into the downstream slope of the pipe. At some specific point, the buoyant and velocity forces become equal and the bubbles remain stationary. If the slope is relatively shallow and long, an air pocket may extend most of the length of the run.

Since the majority of pipelines have flows of varying rates, the self-purging technique cannot be relied upon.

The velocity of flow required to purge a pipe of air has been the subject of numerous experiments. Some of the experimenters have concluded that:

1. The velocity required to move air down a pipe slope increases with pipe diameter and slope.

2. The higher the velocity in a given pipe, the less air binding.

3. The rate of assent of a bubble in a confined tube increases with the tube diameter.

As indicated previously, the locations of air release valves is more important than their size. The various types of air release valves will be described and also the most logical places where air release valves should be installed. It should be emphasized that air carried along by turbulent flow does not conveniently travel at the top of the pipe and readily enter a small opening provided for the installation of an air release valve. Observations of water and entrained air passing through a transparent pipe shows that bubbles are dispersed throughout the fluid. To improve venting, sometimes an air release valve is installed with several connections made into a pipe in a manifold fashion. Another improvement is the installation of air release valves on a cavity or pocket provided on the upper surface of the pipe. This provides a more efficient bubble collecting arrangement.

Types of Air-Release Valves

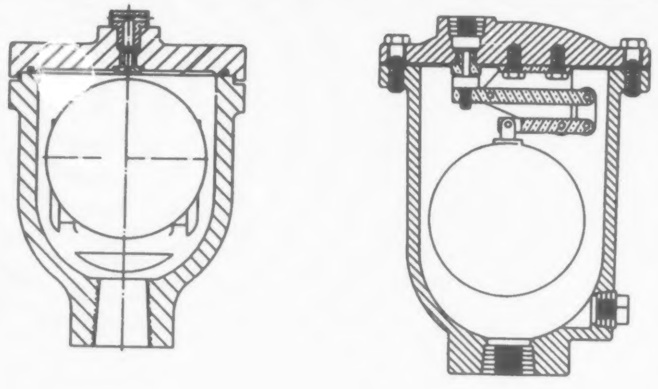
There are just a few basic air release valve designs but countless variations of the basic design. We will describe just the basic designs as the variations are too numerous to elaborate on.

1. Small orifice air release valve.

The purpose of the small orifice is to permit the escape of air from a pipe while the pipe is at the working pressure. The necessity of a small orifice is to permit the opening force of the float or closing element to exceed the force created by the pressure in the system times the area of the orifice. If the orifice were too large, air pressure could easily hold the valve closed. Figure 2 illustrates the general arrangement of small orifice air release valves. The small orifice generally varies between 1/16" to 3/8" depending on the valve design and working pressure. These valves can be provided for pressures beyond 1000 psi.

Figure 2

Small Orifice Air Release Valves



KINETIC VALVE

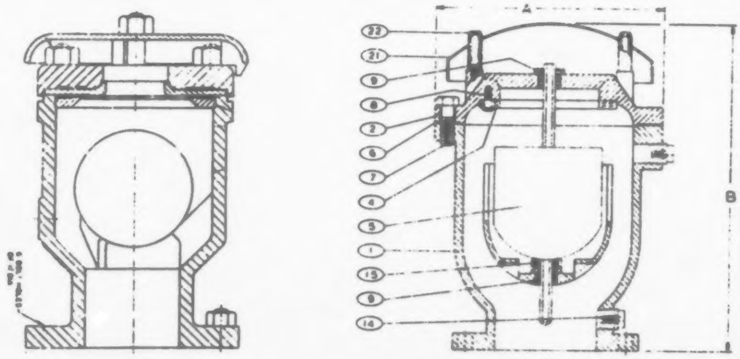
CONVENTIONAL VALVE

2. Large orifice air release and vacuum valves.

The purpose of the large orifice unit is to permit the more rapid escape of air during the initial filling of a line and also permit air to enter the pipe in sufficient volumes during the emptying of the line. This large orifice valve will not permit any further escape of air at the working pressure in the pipe once the valve closes. Due to its large orifice diameter, a relatively low pressure, either air or water, acting on the area of the orifice, is sufficient to support the internal float or buoyant element. Once closed, this valve will not reopen until the system pressure drops to near atmospheric pressure and the float is no longer buoyed. Figure 3 illustrates a large orifice air release and vacuum valve.

Figure 3

Large Orifice Air Release and Vacuum Valve



KINETIC DESIGN

CONVENTIONAL DESIGN

A serious shortcoming regarding large orifice air valves should be mentioned, that is, the problem of blowing closed. During high rates discharge, the float or buoyant element is often caught up in the escaping air stream and slammed to its seat, sometimes collapsing a hollow float. It requires only about 13 psig line pressure to produce an escape of air through an opening at sonic velocity. For this reason, most air release valves must be carefully sized to limit the pipe line pressure to a very low value. This writer has witnessed tests at a national research facility on different valve designs. The findings showed that some designs would blow shut at pressures as low as

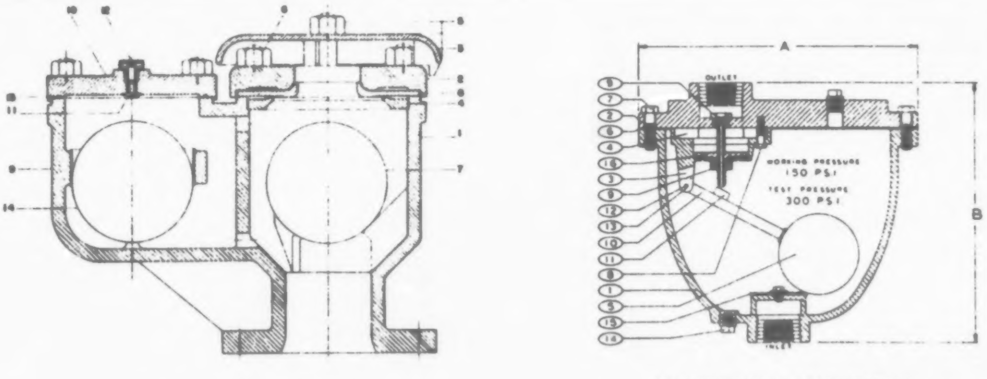
There is only one valve presently available which has been specifically designed not to blow shut while discharging air at any pressure or discharge rate. This valve is designed with the patented "Kinetic" principle. In this design, the aero dynamic forces of the escaping air are utilized to maintain the valve in the full open position even at sonic velocity. This writer has also witnessed tests of this design at pressures over 200 psig and the valve remained open until buoyed closed by the water level. The float or buoyant element seals against a resilient seat when liquid enters the valve.

3. Double or combination air release valves. [See Figure 3A]

This dual valve is designed to perform the functions of both the large and small orifice units. This valve is actually both valves in one valve body. The comments which were made for the individual valves apply to this valve assembly also.

This valve unit is installed where there is a requirement for both large and small orifice valves.

Figure 3A



KINETIC DESIGN
COMBINATION AIR RELEASE & VACUUM VALVE

CONVENTIONAL COMBINATION
AIR RELEASE & VACUUM VALVE

Installation

1. **Peaks.** This is the foremost place where air release valves are required. A peak is located by reference to the hydraulic gradient and not to a horizontal datum line. A peak may be considered as any section of pipe that slopes up towards the hydraulic gradient or runs parallel to it. In the latter instance, an air release valve is required at each end of this run. Double valve units are required at each end of the run and if the run is long,

an air valve should be installed at about 1500 ft to 3000 ft intervals.

A section of the pipeline that is above the hydraulic gradient requires special treatment. Since the pipeline at this location is under negative pressure, air would be drawn into the pipe if an ordinary air release valve were used. On a siphon line, the siphon may be broken. This installation requires valves that permit air to escape but will not permit any air entry into the pipe. Valves of this type are available from manufacturers.

2. Increased downslope. At a point in the pipeline where there is an abrupt increase in downward slope, at least a small orifice unit is required, and occasionally a large orifice unit.

3. Decrease in upward slope. An abrupt reduction in the upward slope can provide a point for an air pocket to accumulate. At this location, at least a small orifice air release valve is required. On occasion, large orifice units may have to be used.

4. Long ascents. Large orifice units generally at 1500 ft to 3000 ft intervals.

5. Long descents. Dual purpose units generally at 1500 ft to 3000 ft intervals.

6. Long horizontals. Dual purpose units at the beginning and end with valves at 1500 ft to 3000 ft intervals. Long horizontal runs of pipe are difficult to ventilate and especially so at lower velocities.

7. Pumps. Air release valves should always be installed on the discharge side of the pump having a suction lift. This valve should be as close to the pump check valve as possible. All air should be expelled from the pump casing as the pump comes up to speed. Often, a time delay relay is employed to delay the opening of a power operated check valve until all of the air has been exhausted. Generally a small orifice unit is adequate but each installation has its own requirements. Too large an orifice can create a problem in the form of a surge. As the water column ascends the pump casing, large quantities of air can pass through a small orifice but the arriving water cannot. This produces an abrupt change in the water column velocity and the resulting surge has been known to damage the pump flanges. In cases where surges are experienced on pump start-up, it has been found that throttling the rate of an air discharge can minimize the problem.

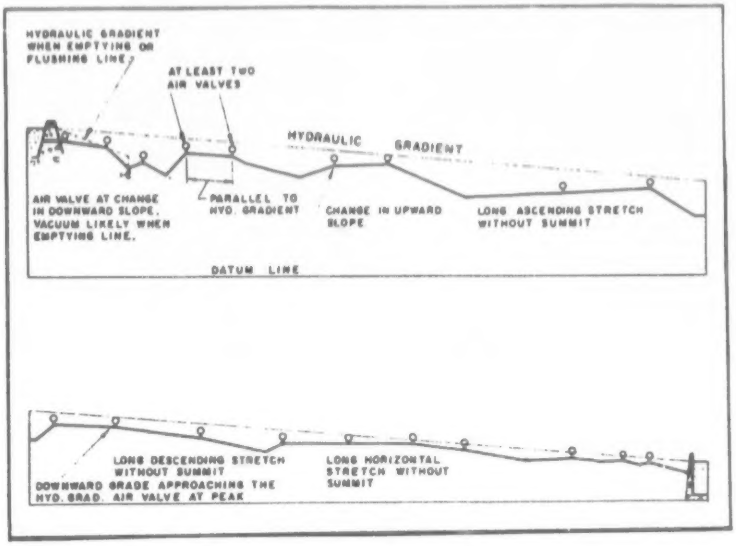
8. Small orifice units should also be installed on high points on large valves, operating cylinders or piping loops. They should also be installed upstream of a reducing taper, orifice plate or similar obstruction. At locations downstream of a pressure reducing device or temperature raising element, small orifice air release valves are recommended.

Sizing

To emphasize again, the proper location of an air release valve is more important than its size. Once the required locations are determined, the following sizing suggestions may be applied.

Figure 4

Suggested Air Release Valve Locations - Typical Pipeline


HYDRAULIC GRADIENT WHEN EMPTYING OR FLUSHING LINE
AT LEAST TWO AIR VALVES
HYDRAULIC GRADIENT
AIR VALVE AT CHANGE IN DOWNWARD SLOPE. VACUUM LIKELY WHEN EMPTYING LINE.
PARALLEL TO HYD. GRADIENT
CHANGE IN UPWARD SLOPE
LONG ASCENDING STRETCH WITHOUT SUMMIT
DATUM LINE
LONG DESCENDING STRETCH WITHOUT SUMMIT
DOWNWARD GRADE APPROACHING THE HYD. GRAD. AIR VALVE AT PEAK
LONG HORIZONTAL STRETCH WITHOUT SUMMIT

Small Orifice Air Release Valves

Since these valves are required to discharge air at pipeline pressures, the pipeline pressure should be determined from the pipeline profile and hydraulic gradient as this pressure will limit the orifice diameter. It must be remembered that the size of the valve is no indication of its orifice diameter. Figure 5 illustrates typical venting capacities of small orifices. For example, a 1" or 2" valve may both have a 3/16" diameter orifice. The valve size only indicates its pipe tap size. To select an exact orifice diameter is complicated by the fact that it is difficult to predetermine the exact amount of entrained air in the flow or how effective the air release valves will be. Also, there may be more than one valve discharging air simultaneously. On systems which occasionally contain water saturated with air, or white water, air release valves may not be very effective. In such cases, the system may require other means of air removal such as pressure control devices and stilling reservoirs to permit the air to escape naturally.

The selection of a proper small orifice diameter is dependant upon many factors but a practical approach appears needed. By referring to our earlier statement that normal water is about 2% air by volume, it is possible to prepare a curve which will indicate a required single orifice diameter which will vent air at that equivalent rate. The curve shown on Figure 6 indicates the required diameter based on flow and pressure.

The orifice diameter arrived at must then be referred to the manufacturer's literature to determine if the required orifice diameter is available for the system pressure involved. If the required orifice diameter is not available, an equivalent number of smaller orifices may be used if the entire venting capability is desired.

Figure 5

Typical Venting Capacities of Small Orifices Coefficient = .7

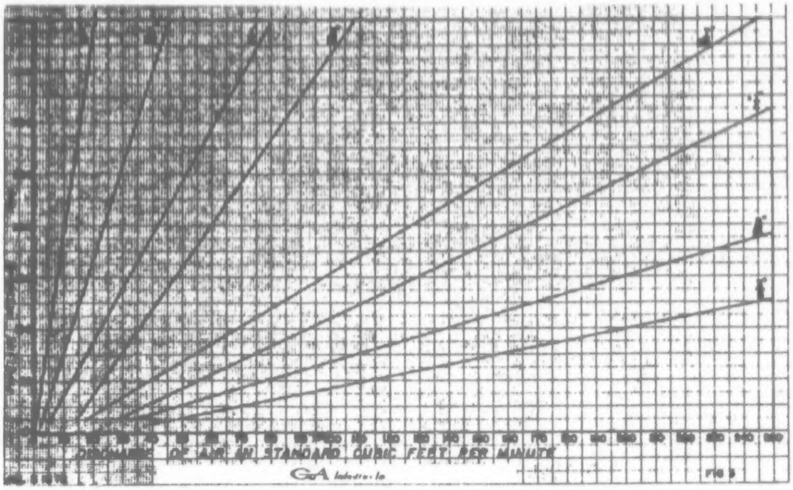
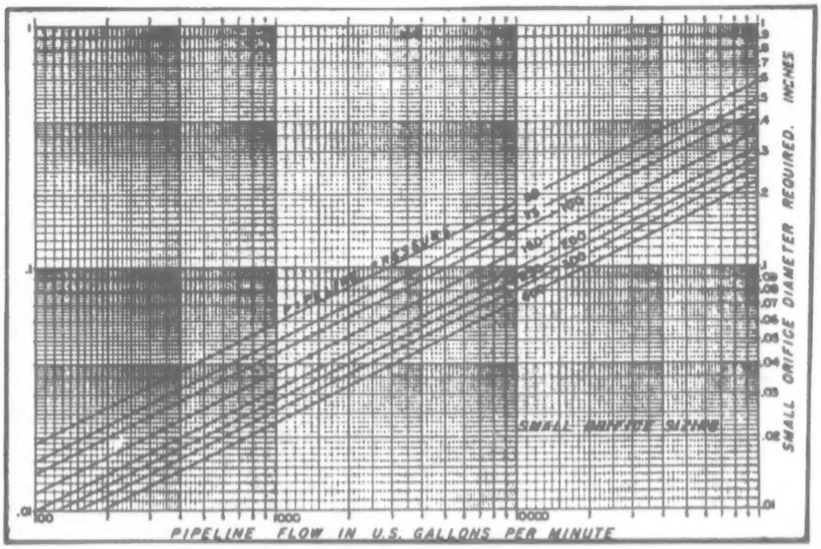


Figure 6



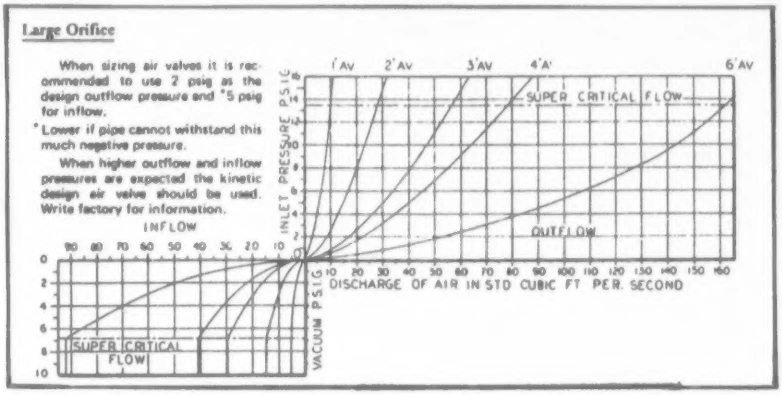
Large Orifice Air Release Valves

The large orifice air release valve is required to vent air during the initial filling of the line. The standard approach to sizing has been to provide adequate valve diameters to vent air at a volumetric rate equal to the pumping rate. By converting the pumping rate to an equivalent cubic feet per minute a chart similar to Figure 7 will indicate the required single valve size. A common design specification is to require more than one valve in the form of a cluster. The specifications would call for no less than two valves or more than four. The discharge capacity is divided among the valves with one of the valves acting as a spare and not considered in the discharge capacity.

It is important to again emphasize the blowing shut tendency of large orifice units. To minimize this problem, valve sizes must be of adequate diameter to limit the pipeline air pressures to very low values, on the order of 1 to 2 psig. Large orifice valves require very gradual filling rates to minimize the water hammer which may result when the air valve is buoyed closed.

With the previous mentioned "Kinetic" design, blowing shut is no longer a problem regardless of the pipeline pressure. This factor offers the additional advantage in that sizing is no longer critical and by maintaining a higher pipeline air pressure during filling the potential water hammer is lessened when the valve closes due to the more gradual filling rate.

Figure 7


Large Orifice

When sizing air valves it is recommended to use 2 psig as the design outflow pressure and *5 psig for inflow.

*Lower if pipe cannot withstand this much negative pressure.

When higher outflow and inflow pressures are expected the kinetic design air valve should be used. Write factory for information.

Vacuum Valves

The large orifice valves previously described will also function as vacuum breaking valves although there are valves specifically designed for just vacuum breaking. Vacuum breaking valves are generally installed at principle high points on a pipeline or at locations where vacuum breaking is required so as to prevent reverse siphoning.

During the emptying of a pipeline, air will have to be admitted. On large diameter thin wall pipes, or perhaps wood stave pipes, air must be admitted to prevent collapse of the pipe. Collapsing is generally not a problem with typical diameters as most pipes can withstand the external pressure of atmosphere plus the earth loading.

The selection of the required vacuum valve diameter or equivalent smaller sizes is based on the rate at which a cavity is being formed either by way of a line break, draining of the line, or as the result of a water column separation. To indicate how a vacuum valve diameter is selected we will assume a typical pipeline where a change in the pipeline profile occurs. Refer to Figure 7c. By also assuming that at the change in slope, the upper portion of the pipe has a gradual slope while the lower section of the pipe has a steeper slope, both in the same direction. Should the line break at the bottom of the steeper slope, water will flow down the steeper slope more quickly than down the upper, more gradual slope. At the point of the change in slope, a cavity will form equal to the differences in these two flows. The flow in each section of pipe may be calculated by the "Chezy" or "Williams and Hazen" formulas or the chart Figure 7a may be used. By referring to a chart similar to Figure 7 for inflow, a required valve size or sizes may be found. The negative pressure value used may be governed by the external pressure limitations of the pipe. It will be noted that at about minus 7 psig, air enters the pipe at sonic velocity and there will be no further increase in the inflow even though the pipe may be at a greater vacuum.

Figure 7c

Vacuum Valve Locating Example

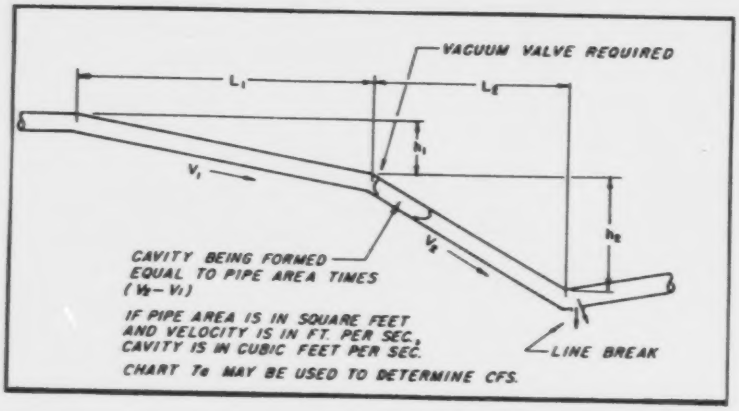
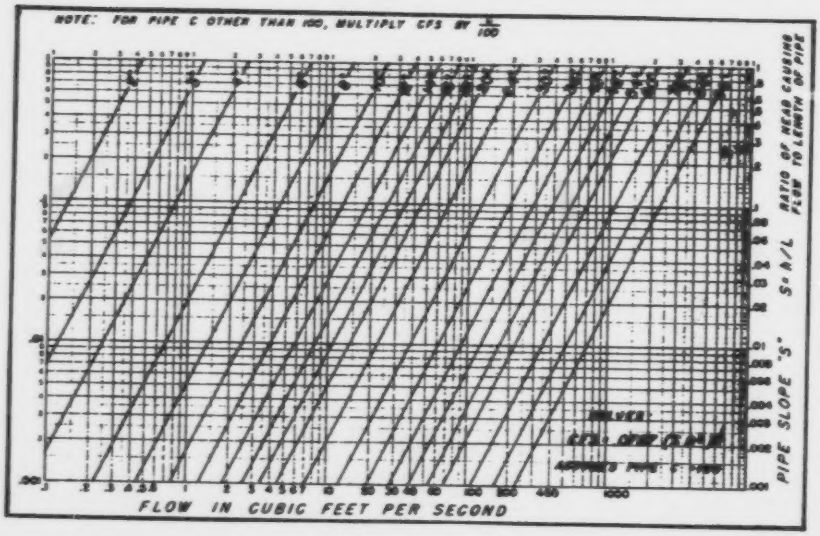

VACUUM VALVE REQUIRED
L_1
L_2
h_1
h_2
V_1
V_2
CAVITY BEING FORMED EQUAL TO PIPE AREA TIMES ($V_2 - V_1$)
IF PIPE AREA IS IN SQUARE FEET AND VELOCITY IS IN FT. PER SEC., CAVITY IS IN CUBIC FEET PER SEC.
CHART 7a MAY BE USED TO DETERMINE CFS.
LINE BREAK

Figure 7a



The problem previously mentioned with large orifice valves is again apparent with vacuum valves. That is, with the blowing closed problem in large orifice vacuum valves, the float may be drawn up towards the inlet opening by the inrushing air. The problem here is not as severe since the float cannot completely stop the inflow but can offer some restriction. Some vacuum valve designs require perhaps only a 5 psi allowable pressure differential across the valve on inflow. The "Kinetic" valve design does not have this shortcoming and will operate with a complete vacuum within the pipe.

Installation

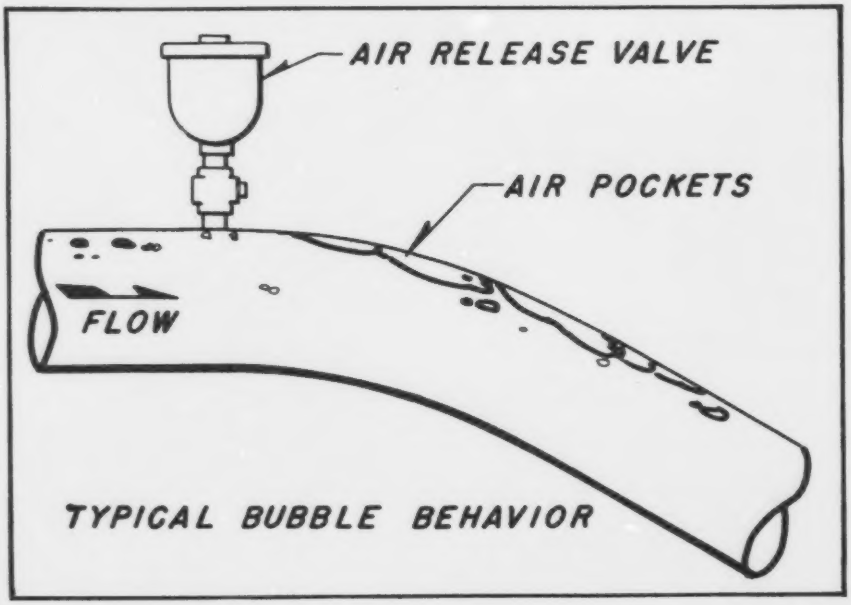
All air release valves should be considered as automatic equipment and periodically inspected. The valves would best be installed in a valve vault and in freezing areas, below the frost line. It is important that the valve vaults be properly ventilated and drained. An improperly vented vault may become pressurized during the discharge of air. During periods of inflow, the vault may be subjected to severe negative pressures. Inadequate draining may result in a flooded vault with a possible health hazard. Outside vents should be screened to discourage the entrance of birds or other creatures.

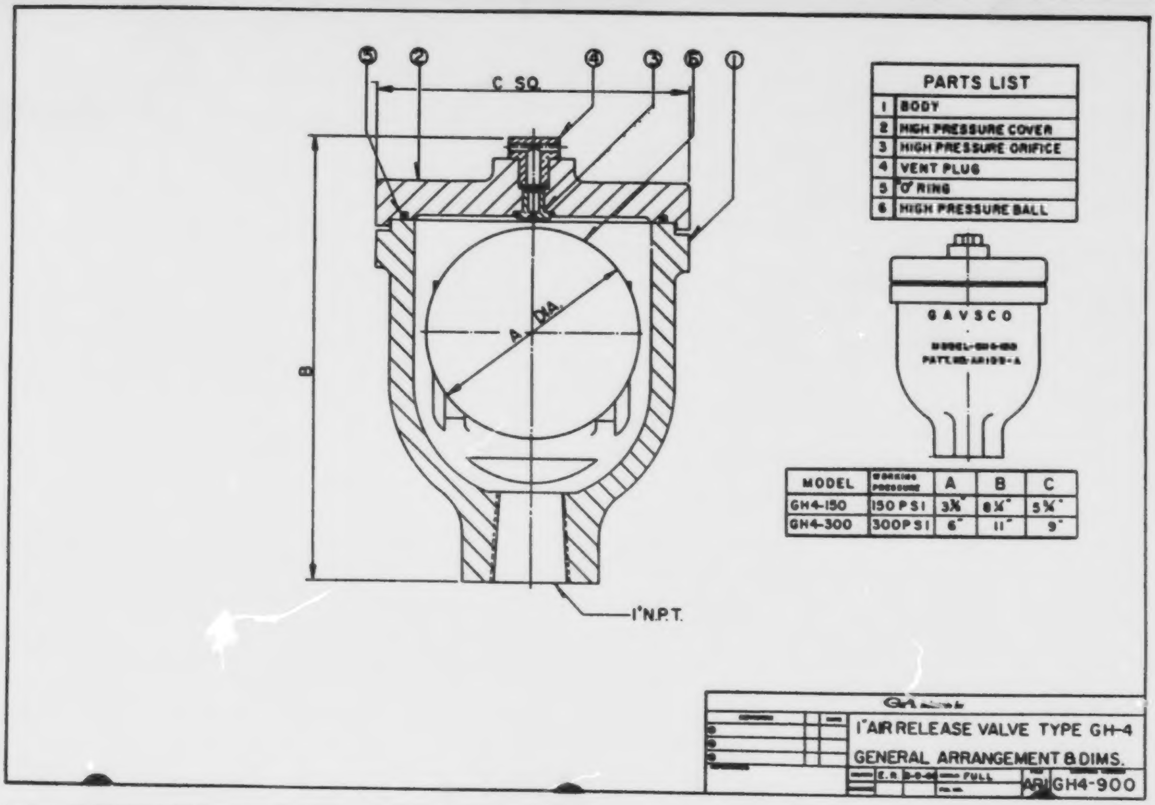
In all installations, it is suggested that an isolating valve be installed under the air valve to permit inspection and servicing.

After considering the comments in this paper, a designer, in the interest of economics, may wish to minimize the number of required air release valve locations. One method of doing this would be to reexamine the pipe invert grades and reslope certain pipeline sections.

It was not the intent or the ability of this paper to offer solutions to all air relieving problems. It was intended to increase the awareness of the reader with some factors associated with air release valves and their application.

The careful application of air release valves becomes an investment bringing immediate returns without major cost considerations.



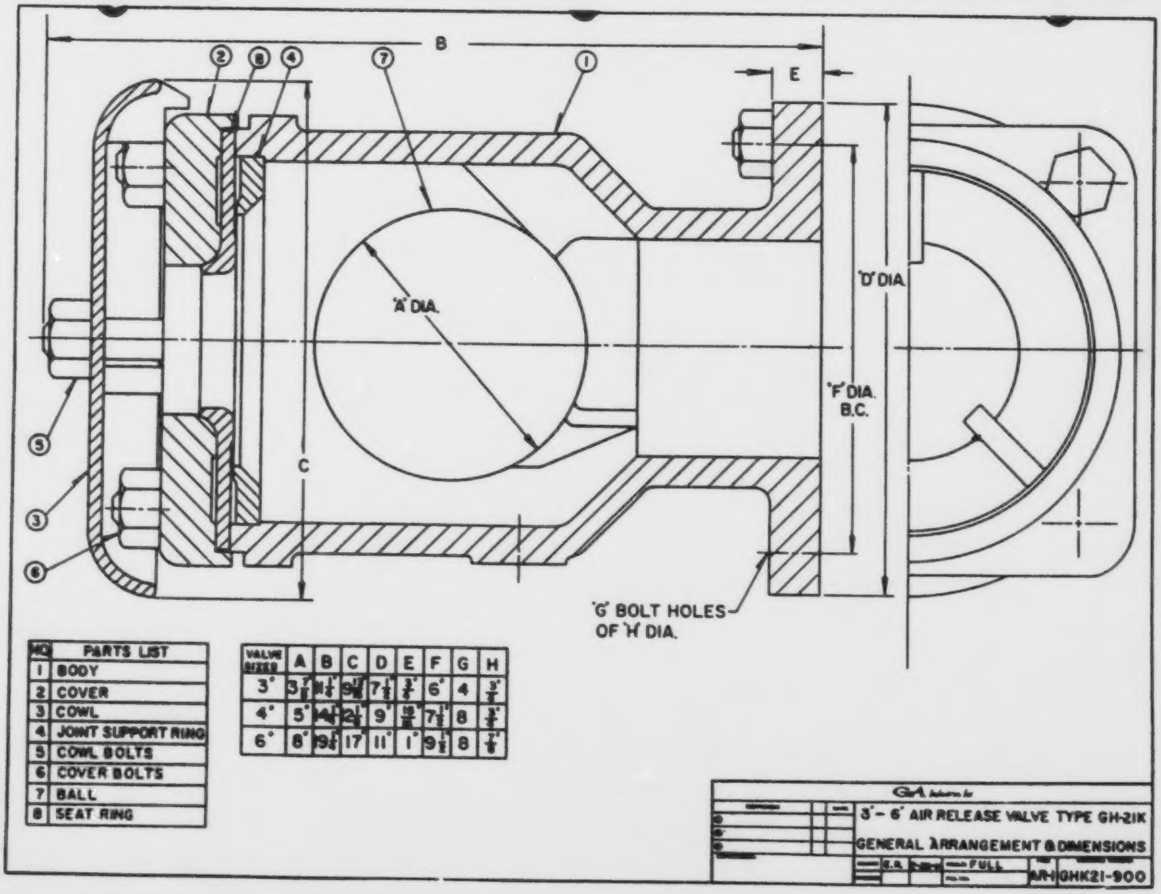

⑤
②
④
③
⑥
①
C SQ.
B
A DIA.
1" N.P.T.
GAVSCO

	PARTS LIST
1	BODY
2	HIGH PRESSURE COVER
3	HIGH PRESSURE ORIFICE
4	VENT PLUG
5	"O" RING
6	HIGH PRESSURE BALL

MODEL	WORKING PRESSURE	A	B	C
GH4-150	150 PSI	3⅜"	8¼"	5¾"
GH4-300	300 PSI	6"	11"	9"

GA Industries Inc.

1" AIR RELEASE VALVE TYPE GH-4
GENERAL ARRANGEMENT & DIMS.
E.R. FULL
AR-1 GH4-900

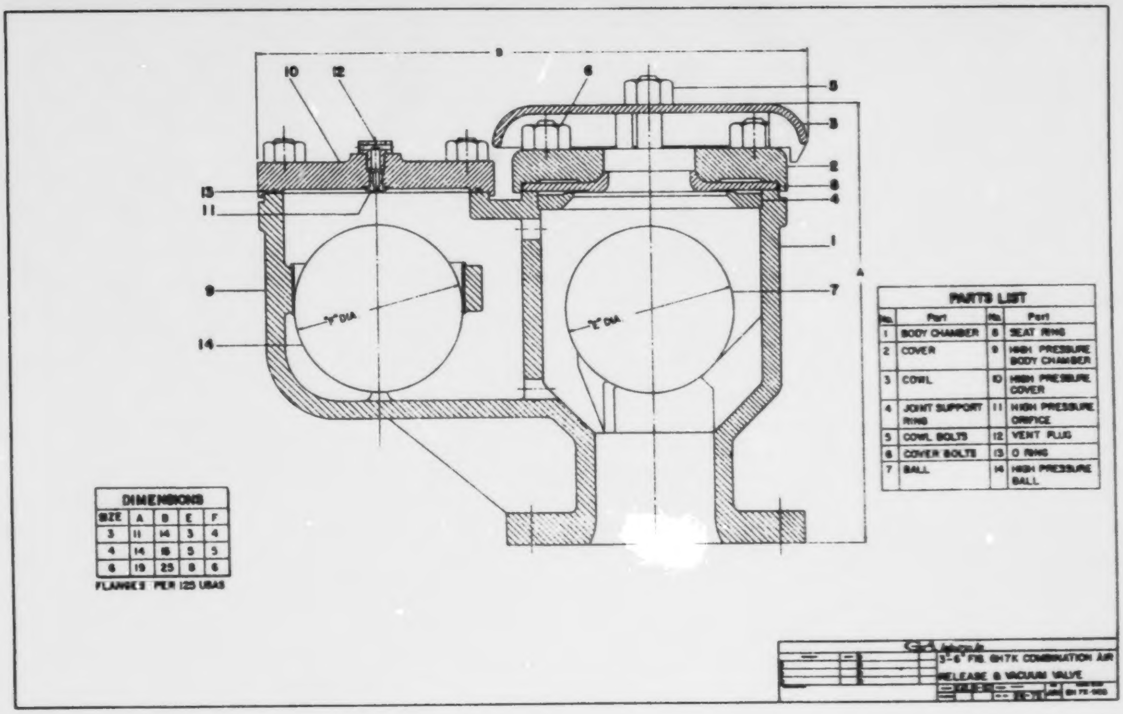

B
② ⑧ ④
⑦
①
E
"A" DIA.
"D" DIA.
"F" DIA. B.C.
C
⑤
③
⑥
"G" BOLT HOLES OF "H" DIA.

NO.	PARTS LIST
1	BODY
2	COVER
3	COWL
4	JOINT SUPPORT RING
5	COWL BOLTS
6	COVER BOLTS
7	BALL
8	SEAT RING

VALVE SIZES	A	B	C	D	E	F	G	H
3"	3⅞"	11¼"	9[illegible]"	7½"	¾"	6"	4	¾"
4"	5"	14[illegible]"	12¼"	9"	15/16"	7½"	8	¾"
6"	8"	19[illegible]"	17"	11"	1"	9½"	8	⅞"

GA Industries Inc.

3" - 6" AIR RELEASE VALVE TYPE GH-21K
GENERAL ARRANGEMENT & DIMENSIONS
E.R. FULL
AR-1 GHK21-900



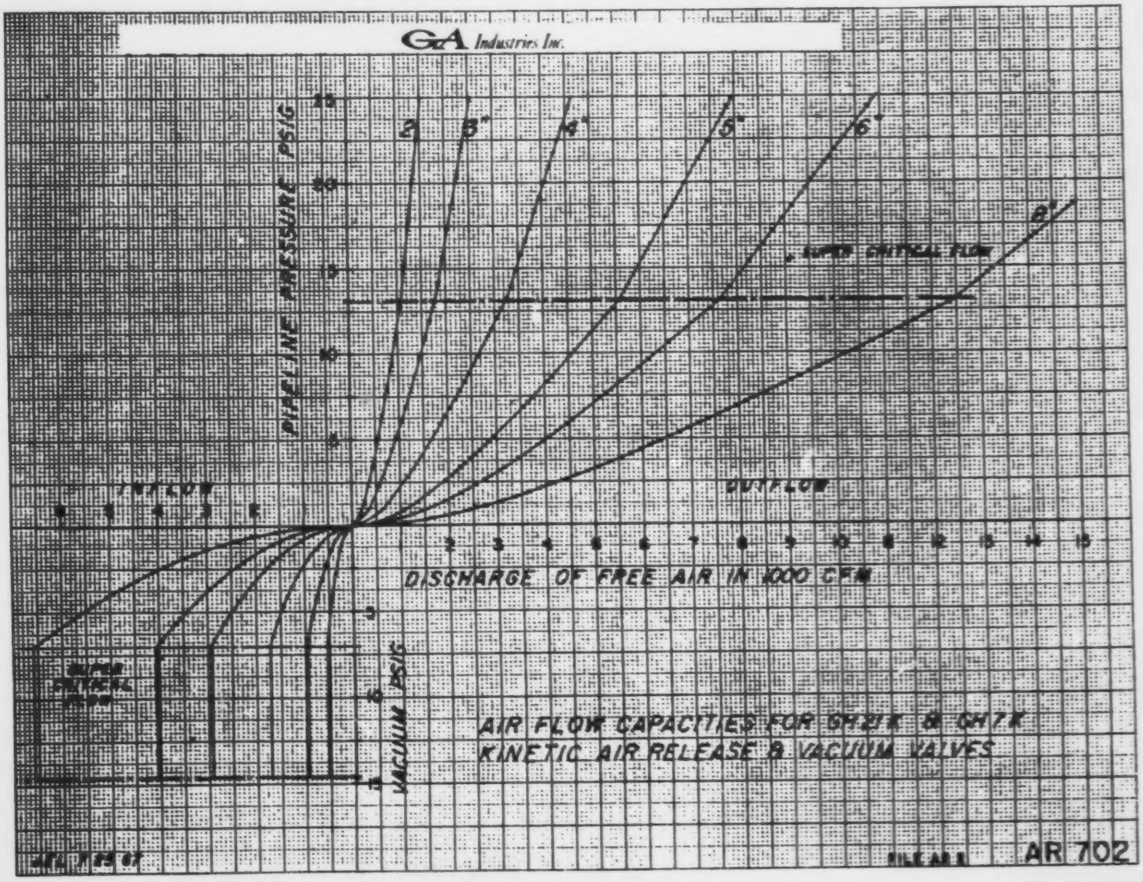
PARTS LIST			
No.	Part	No.	Part
1	BODY CHAMBER	8	SEAT RING
2	COVER	9	HIGH PRESSURE BODY CHAMBER
3	COWL	10	HIGH PRESSURE COVER
4	JOINT SUPPORT RING	11	HIGH PRESSURE ORIFICE
5	COWL BOLTS	12	VENT PLUG
6	COVER BOLTS	13	O RING
7	BALL	14	HIGH PRESSURE BALL

DIMENSIONS				
SIZE	A	B	E	F
3	11	14	3	4
4	14	16	5	5
6	19	25	8	6

FLANGES PER 125 USAS

G-A Industries

3–6" FIG. GH7K COMBINATION AIR RELEASE & VACUUM VALVE



AIR FLOW CAPACITIES FOR GH21K & GH7K
KINETIC AIR RELEASE & VACUUM VALVES

AR 702

Sewage Force Mains

In the previous section on air release valves for water systems, it was shown that there are two requirements for air release valves. On occasion, air needs to be admitted into a pipeline, and on these occasions air must be vented from a pipeline. In sewage systems, the problem is more difficult because of the nature of the fluid being handled. If raw sewage were permitted to enter an air valve such as those in Figure 2, sewage would soon plug the small orifice or foul the lever mechanism rendering the air valve ineffective. Sewage air valves are designed so that the valves close with an air cavity remaining within the valve. The fluid under pressure will compress this air to a pressure equaling the fluid pressure, but the sewage level within the valve does not reach the mechanism or orifice. This requires the air valve to become long, also requiring higher valve vaults. For example, the desired volume of air under 125 psi working pressure within the air valve body can only be about one tenth the air volume within the air valve with no fluid in it. Most sewage air release valves can be provided with a flush-out attachment whereby the valve mechanism can be flushed with a water hose when required.

We will review the requirements for large and small orifice air release valves in sewage systems.

Large Orifice Units

During the initial filling of a pipeline, the air within the pipeline must be vented. This normally requires a large orifice since a small orifice may require excessively long venting times. As the pipeline is filled, the large orifice air release valves will close, and remain closed - perhaps forever, or until the pipeline is drained. The large orifice valve cannot release under working pressures. Large orifice air valves also serve as vacuum breaking valves by admitting air in large volumes during negative pressure conditions within the pipe.

Sizing Large Orifice Units

A typical procedure for sizing the large orifice would be to determine the pipeline flowing rate and convert the gallons per minute or MGD rate to cubic feet per minute or seconds and refer to a discharge capacity chart for a particular air valve design, and opposite the pipeline pressure scale, read a required orifice diameter or valve size. The pressure selected for this venting capacity must not exceed about two to four PSIG because of the "blowing shut" possibility of all but one air release valve design. This one valve that cannot be blown shut is the "Kinetic" design valve. The pipeline pressure is of no concern since air flows at sonic velocity, in or out, cannot blow the "Kinetic" valve closed.

When a required orifice diameter is found, under critical pipeline conditions where pipeline collapse must be avoided, the equivalent air valves may be specified as no less than two units or more than four units with one unit as a standby.

The orifice diameter or diameters selected for admitting air will be more than ample for venting air during the initial filling of the pipeline because the pipeline is usually filled slowly and not at the maximum flow rate. Slowly filling the pipeline is strongly recommended to avoid surging problems. See Figure 4 for locating of valves on the pipeline.

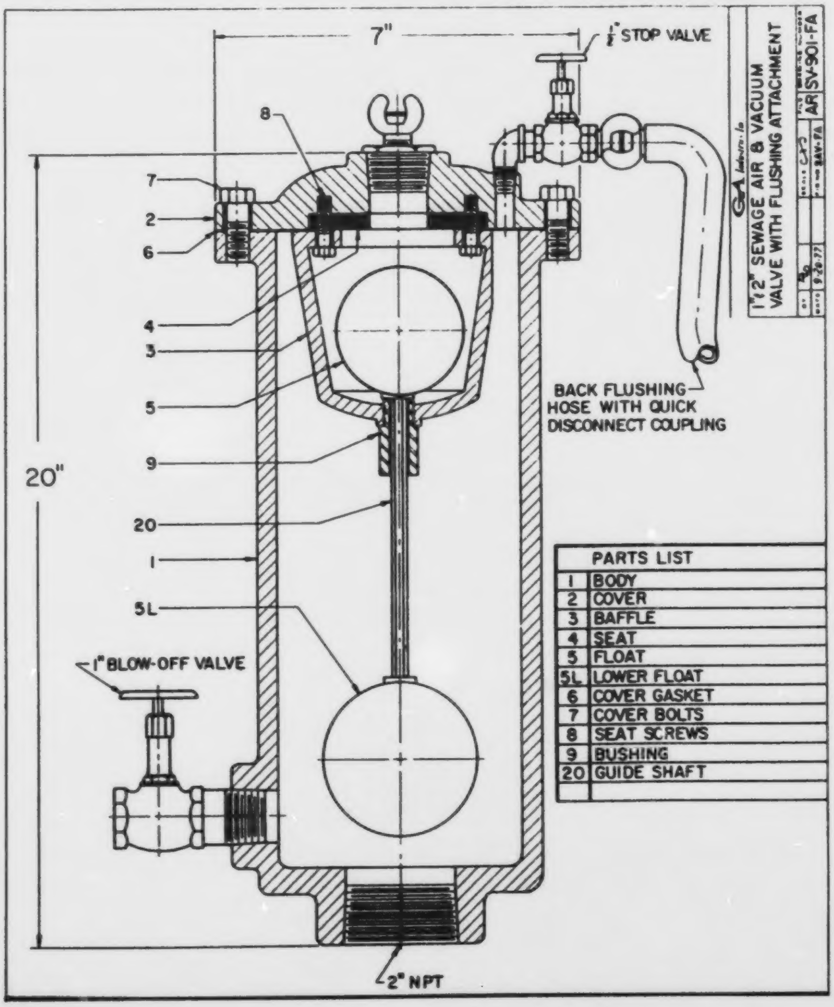
Small Orifice Units

Everywhere a large orifice unit is installed, a small orifice valve must also be installed adjacent to the large orifice valve. Conversely, a large orifice unit need not necessarily be required everywhere a small orifice unit is installed.

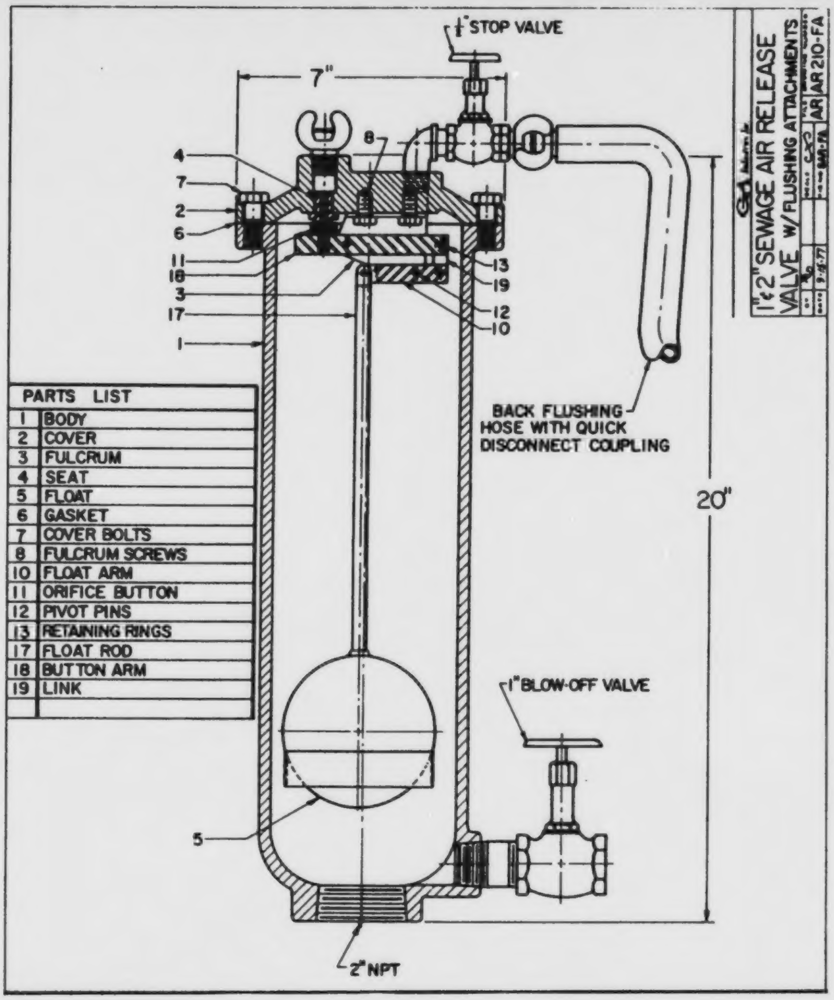
Large orifice units are usually at principle high points in the pipeline, whereas small orifice units are also installed at low areas in the pipe where abrupt changes in pipe slope occur. Figure 4 illustrates preferred valve locations.

All small orifice air release valves have small orifices so sizing is generally not as important as large orifice units. Small orifice units vent air as air accumulates and valve locations are much more significant than their orifice size.

It is easy to understand an engineer's desire to use as few valves as possible because of the expense of valves and valve vaults. However, the overlooking of a single air bubble results in a reduction of the cross sectional flow area of the pipe with the result that the pump must develop a slightly higher head. Therefore, the real economics is in installing adequate numbers of air valves.


7"
½" STOP VALVE
8
7
2
6
4
3
5
9
20
1
5L
20"
1" BLOW-OFF VALVE
2" NPT
BACK FLUSHING HOSE WITH QUICK DISCONNECT COUPLING
G-A Industries, Inc.
1½" SEWAGE AIR & VACUUM VALVE WITH FLUSHING ATTACHMENT
AR
SV-901-FA
SAV-FA
9-20-77

	PARTS LIST
1	BODY
2	COVER
3	BAFFLE
4	SEAT
5	FLOAT
5L	LOWER FLOAT
6	COVER GASKET
7	COVER BOLTS
8	SEAT SCREWS
9	BUSHING
20	GUIDE SHAFT



1/2" STOP VALVE
7"
8
4
7
2
6
11
18
3
17
1
13
19
12
10
BACK FLUSHING HOSE WITH QUICK DISCONNECT COUPLING
20"
1" BLOW-OFF VALVE
5
2" NPT
1" & 2" SEWAGE AIR RELEASE VALVE W/FLUSHING ATTACHMENTS
AR
AR210-FA
9-12-77

PARTS LIST	
1	BODY
2	COVER
3	FULCRUM
4	SEAT
5	FLOAT
6	GASKET
7	COVER BOLTS
8	FULCRUM SCREWS
10	FLOAT ARM
11	ORIFICE BUTTON
12	PIVOT PINS
13	RETAINING RINGS
17	FLOAT ROD
18	BUTTON ARM
19	LINK

SEWAGE AIR VALVES

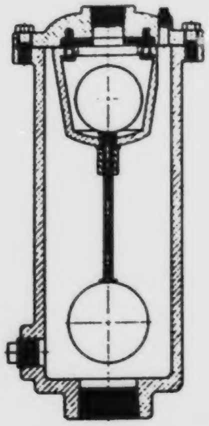


FIG. SAV

LARGE ORIFICE
SEWAGE AIR RELEASE
AND VACUUM VALVE

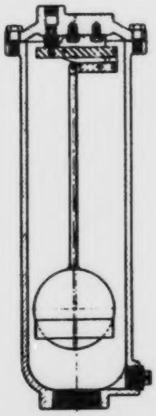


FIG. SAR

SMALL ORIFICE
SEWAGE AIR RELEASE VALVE

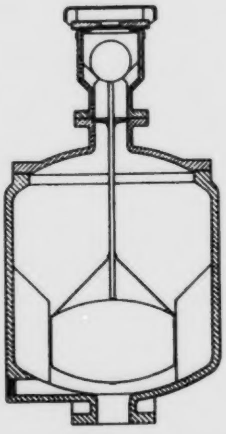


FIG. K-SAV

LARGE ORIFICE KINETIC
AIR RELEASE & VACUUM VALVE

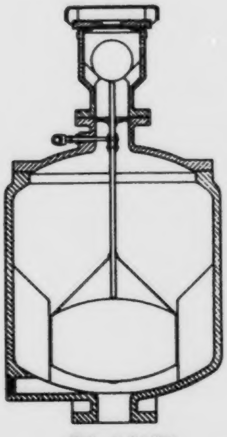


FIG. K-SARV

COMBINATION LARGE &
SMALL ORIFICE AIR RELEASE
AND VACUUM VALVE WITH
KINETIC FEATURES

METER CHANGE-OUT PROGRAMS
FOR
LARGE CUSTOMER METERS
SOURCE METERS
RESIDENTIAL METERS

by

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Customer metering practices for the Water Utilities in the United States, by and large, have neglected a substantial resource for additional income by not developing within their utility a thorough meter maintenance and meter operational surveillance program. For the past 10 years, Community Consultants Inc. has been doing research on the accuracy of various sizes and customer group types of meters in the utilities throughout the United States.

We now estimate that the average daily loss in revenue to water utilities in the United States, due to unaccounted for water, exceeds $2,000,000 a day or approximately $800,000,000 a year. Our research data base is based on testing of several thousand customer meters in place (without removing the meter from the service) and from gathering research data through the American Waterworks Association Seminar on Water Loss Reduction and Revenue increases which we have taught throughout the United States.

The cause of this revenue loss is as follows:

1. Improper meter sizing.
2. Improper meter installation.
3. Improper meter selection.
4. Lack of meter testing and repair programs.
5. Not knowing the meter's performance based on the customers' usage rate requirements.
6. Not having a meter surveillance program for comparison of monthly customer usage for the same month in previous years.

When an effective meter installation maintenance and surveillance program is established, significant revenue increases occur. It is important to emphasize the simple concept that when an inaccurate meter is brought into accuracy, that the revenue increase occurs at the time of meter repair or replacement. We have been involved with the City of Phoenix in a Large Meter Maintenance Program. The 1984 revenue recovery for the City of Phoenix was $1,167,000. That is revenue which was not realized in the previous year due to an ineffective meter maintenance program. In a utility with approximately 12,000 connections, an increase of $270,000 a year was realized by establishing an effective meter maintenance program for the larger users within the water utility. It is common within a water utility that 50% of the water will be sold to 10% of the customers; therefore, it is essential to develop a revenue recovery program to concentrate the efforts of proper meter performance on the 10% who are the largest users.

Our firm has assisted the California Department of Water Resources in teaching seminars to approximately 65 cities on the subject of Water Auditing and Leak Detection. In the Water Auditing phase, the utilities were expected to test the majority of the master source meters as well as a sample testing program of their customer meters. Report after report came in showing significant revenue increases through the meter maintenance programs. One utility is reporting an increase in revenue of $480,000 annually for the first year expenditure of $8,000. This relates to a 60-1 benefit-cost ratio (see the Department of Water Resources, February 1985, report). It is not uncommon to realize a 15-1 benefit-cost ratio in a water utility. When the program is sustained the first year, the benefit-cost ratio can average approximately 30-1.

By following a four-point plan for large meter maintenance and surveillance, significant revenues will be recovered.

A FOUR-POINT PLAN

1. MAKE A METER SURVEY AND INSPECTION OF EACH METER
    A. Is there a test plug?
    B. Is there a bypass?
    C. Are there isolating valves on both sides of the meter?
    D. Is the meter installed properly?
    E. Is the correct type of meter installed?
    F. Retrofit meters that are inadequately installed.

2. TEST THE METER FOR ACCURACY
    A. Begin the test from 0 and test through the meter's capacity.
    B. Repair if necessary.
    C. Change the meter or retrofit if not repairable.

3. RECORD THE USAGE RATE FOR 24 HOURS
    A. Compare the historical usage rate with the test accuracy results.
    B. Perform a 24-hour (or more) continuous recording of meters that are suspected of being missized or of malfunctioning.
    C. Review the usage rate at the change-over point with the accuracy of the meter at that flow rate.
    D. Change the meter size or retrofit if necessary.

4. PREPARE A LONG-TERM SURVEILLANCE PROGRAM
    A. Manual surveillance.
    B. Computer surveillance.
    C. When the meter performance is not consistent, the meter will be inspected for:
        1. A change in customer usage rates.
        2. Meter malfunction.

The first point suggests that we make a meter survey and inspection of each meter. In performing this inspection, we are going to find many large meters that are obsolete and parts that are no longer available for meter maintenance. Many utilities are taking out the typical compound meter and replacing it with Turbo type meters. If a utility is in this type of program, may we caution the utility to examine the customers usage rate where the meter is being changed out to determine the following:

1. What is the minimum usage rate the customer demands for service?
2. What is the average usage rate the customer demands for service?
3. What is the maximum usage rate the customer demands for service?

In evaluating this process, many times we find the meter was sized improperly because it was sized for the nominal size of the service line. The design criteria for the service are not the design criteria for the meter. The service line is designed to provide the customer with the maximum rate of flow that they may require in any condition, including future expansion. The frequency of this maximum rate of flow might be very seldom. The criteria for sizing the meter should be based on the customers' anticipated usage rates ensuring that the meter will register accurately for all flow rates at which the customer anticipates using water.

We have historically expected the manufacturers to furnish our large meters with testing facilities, such as test plugs. The new Turbo type meters do not have test facilities. We recommend that test facilities be installed and provisions be made in every installation to provide for testing of the meter through the complete flow range, that is from 0 to the maximum capacity of the meter.

We feel that it is essential to record the usage rates through an accurate service meter. The flow rates that the customer demands on the system for 24 hours, 48 hours or extended periods of time, are used to evaluate the customer's usage rate and the meter's performance based upon the customer's usage rate. In the meter change-out program, it is essential to establish good data to determine the size and type of meter that should be installed to provide accurate registration of water usage over the full range of flows demanded by the customer. In many installations we find that a meter sized smaller than the service line will adequately serve the customers' peak flow demands and also ensure that the meter will more accurately register all of the water delivered to the customer.

Phase 2 of the Four Point Plan is the testing phase based on the flow demands that the meter technician puts on the meter. Phase 3 records the flow rates which the customer puts on the meter. In the case of Turbo meter installations, we want to ensure that the meter is going to register accurately at the low flows as well as at the high flows.

We have found significant problems with the magnetic coupling on some Turbo meters failing at moderate rates of flow. The magnetic coupling problem is going to be an extremely significant problem in revenue losses in the future. The basic concept is that whenever the mechanical evergy required to operate the meter drive shaft and register exceed the energy available through the magnetic coupling, a magnetic coupling failure will result. Many factors affect the magnetic coupling. Some of these are as follows:

> The friction of drag is placed upon the mechanical operating parts of the meter from the magnetic coupling to the odometers in the meter register. Bearing misalignment and malfunction are causes for the increased energy demand. The speed at which the meter operates also increases the energy required. The magnetic field of the magnet is affected by the plate which separates the drive and driven magnets.

As the rotational velocity of the drive magnet increases, the available energy to the driven magnet through the plate is decreased until the magnetic couple begins to fail. The meter becomes less accurate at high rates of flow. Any chemical buildup or collection of foreign objects to the magnets also affects the performance of the magnets.

Our research shows that a 3-inch meter whose design capacity should take the meters performance to 600 gallons per minute, had a magnetic coupling failure at 200 gallons per minute. A 4-inch meter whose operating range should be 700 gallons per minute, had a magnetic coupling failure at approximately 250 gallons per minute. The point of failure changes with age and circumstances of the meter. A sudden velocity change also adversely affects the magnetic coupling. Our research indicates that Turbo meters which have a reduction in revolutions of the Turbo to the Magnetic Coupling have lower failure rates than do those which have couplings that are turning on a one-on-one ratio with the Turbo.

Another significant problem which we find in our larger meter installations is with the single registering compound meter. This is where the low flow measuring element is registered on the same register as the turbo, turbine or high flow rate side of the meter. A single registering compound, like any other compound, has four basic operating components: a register, a low flow measuring element, a check valve and a high flow measuring element. In the event of a failure, whether it is partial or total of either measuring element, or a failure of a check valve to close, the single register will show consumption. Unless a surveillance program has been established, the utility will not recognize the change in usage rate for the particular month or billing cycle of the year. For meters that have been in service, it would be necessary to test the meter for accuracy and record the monthly usage rate to begin establishing a usage pattern. The surveillance program can be accomplished by incorporating this into a utility's computer programming software. If you do not have access to a computer, this can be done manually.

Whichever way you choose to establish the surveillance program, a data base should be developed so that each month of the year can be compared to the corresponding month of at least three of the previous years. Within each month, there is a plot established for each year of a three-year program graphed out for that same month. For example: January would have a bar graph for 1985, 86 and 87. A simple visual observation of the graph can establish the meter's performance based on the previous years. A high-low limit established for each billing cycle of the total year is not adequate because most meters lose accuracy at a gradual rate.

An example is shown here where a 6-inch single registering compound meter was recording adequately to provide income of approximately $500 to $600 per month. However, when the meter was tested and repaired and recalibrated, the increased revenue totalled $76,000 per year for an increase of more than $6,000 per month.

Part 4 of the 4-point plan outlines the criteria for a surveillance program. When we see a divergence, either high or low from the previous year during the same month, it alerts us to the meter's performance. We should evaluate the meter which may require a field inspection and testing to determine why the change has occurred. The change can occur for two basic reasons.

1. That the customers usage rate has changed.
2. That the meter has begun to fail and lose its performance accuracy.

In our meter change-out programs, it is essential that we establish a program that will provide the highest level of observation for meter performance and sustain the highest possible levels of accuracy on the large usage customers. We must also ensure that in our new Turbo meter installations, we provide in meter installation the following:

1. Provisions to ensure the meter will register at low flows. Turbo meters can be fitted with a weighted check valve and bypass if measurement at low flows is required. There are some nice features in this installation in that it is a compound metering situation but we can vary the size of the low flow meter depending on the individual customer installation needs.

2. Be sure isolating valves and bypass are installed so that the meter can be serviced and maintained without taking the customer "out of service."

3. Make sure there is a metered bypass to provide the customer with water during meter maintenance. In a situation where we have a Turbo meter with a metered bypass, it is very important to ensure that this installation provides for the independent testing of each meter. This procedure allows the customer to be served with the metered service during the maintenance, testing and recalibration.

MASTER SOURCE METERS

Many master source meters, or the meters that measure the water coming into our utility, are not operating properly. One of the most significant problems we experience is when the flows coming into the system are less than the accurate operating range of the meter. We experience this problem with differential pressure meters, flow tubes, orifice plates, etc. Ultrasonic and electromagnetic meters are also vulnerable to similar inaccuracies primarily because the velocities are so low that the meter is not designed to register accurately at those flows. Propeller meters generally have fairly high minimum rates of flow; therefore, we find many utilities have water production records which are not accurate because the meters do not operate at the rates of flow produced by the source. It is essential to determine what rate of flow will be produced by a source when sizing and selecting the type of meter to register incoming flows. This is a significant problem where treatment facilities are designed for future rates of flow and existing rates of flows do not meet the minimum rate of flow for the designed meter.

Provisions for testing source meters with velocity or differential pressure meters should be installed. It is important to have the approximately 5 pipe diameters downstream and 10 pipe diameters upstream prior to a testing device installation to ensure proper velocity profiles are created at the point of comparative testing. Here again, the comparative meter will not operate accurately at velocities with less than approximately 1 foot per second. Differential pressure meters used for comparative testing will not function at less than 20 PSI.

RESIDENTIAL METERS

There is a disparity throughout the country on the size of meter to install in a customer service. We are seeing more appliances using water in the homes which are requiring higher rates of flow. However, we must realize that the restriction in the rate of flow is generally not the size of the meter but the headloss in the service line from the main to the point of use within the home. In testing residential meters, we find that the typical 1-inch service line has a capacity of less than 20 gallons per minute. A typical 3/4-inch service line has a capacity of less than 13 gallons per minute. The 1-inch meter is rated for 50 gallons per minute. The 5/8 x 3/4-inch meter is rated for 20 gallons per minute. Extensive research shows that in circumstances when there is not outside watering, 90% of the water will be delivered to the home at less than 3 gallons per minute. Our research indicates that a 1-inch meter servicing a duplex, registered 4700 gallons in 24 hours with a flow rate that never exceeded 17 gallons per minute for extended periods of time. The flushing of our lavatories requires less than one gallon per minute to refill the storage reservoir.

We recommend that a utility consider installing a 1-inch service line with a 5/8 x 3/4-inch meter to ensure the highest accuracy levels at the low rates of flow. The meter will still have the capacity to deliver water at the higher rates of flow.

We recommend periodically recording water usage through residential meters. This recording can be done simply by installing a double yoke meter assembly which fits into the typical meter yoke assembly and will set two meters simultaneously. One metering unit will be the customer's meter. The second metering unit would be a very precisely tuned meter which records accurately at low rates of flow and is capable of driving a chart recorder. The comparison here is twofold:

1. A recording of the customers usage rates to determine at what rates the customer is using water at various times of day.

2. The total flow through the highly tuned meter can be compared to the accuracy of the customer's meter based on the various flow rates demanded by the customer.

In the selection of meter type and brand, we recommend that the utility perform a random meter testing program. The meter testing program evaluates the accuracy of each meter manufacturer, type and style whether it be a positive displacement disk or piston, or a velocity multi-jet type meter. We recommend that a broad enough base be established in the testing of these meters to establish criteria based on the total water registered on the meter, and the age of the meter. The research data-base will help you establish which meter [positive displacement, multi-jet, brand or style] is giving the best performance based on the following criteria:

> The meter selected should give the utility the highest rate of accuracy over the longest period of time at the lowest maintenance cost.

We recommend that utilities with the old gear drive stuffing box meters consider a meter replacement program.

Many older gear driven stuffing box meters still have high performance but the availability of parts, the age of the measuring element, the wear of the gearing assemblies and the increased resistance or drag will affect the meter's accuracy. The meter will begin losing its accuracy at the lower rates of flow and will progressively become inaccurate at the higher rates of flow until the meter stops completely. When entering into a change-out program, test several meters by using a double yoke meter assembly putting the old customer meter in on one side and the new meter to be evaluated on the other side to determine the difference in total registration over a designated period of time. This can also be done in evaluating a new meter's performance by placing the new meters that are under consideration in the double yoke assembly and determining the total registration on the meter over a period of time by comparison. By this method, meter performance in actual operating conditions can be evaluated.

We recommend that very little consideration be given to low bid on meters when selecting the meter which you are going to purchase. We recommend that meter selection be based on the research that you do within your utility. Meter guarantees and future meter maintenance replacement programs are not as important as the initial selection of the type of meter that will function best within your utility and the characteristics of the water which you are producing and distributing.

A recent issue of Mainstream magazine indicated the result of research done on the needs for water utilities. There is no question that revenues are the most critical and are rated number one as short-term and long-term needs. We have no objection to this concept, but we need to realize a greater revenue from meter sales. We feel that the first step in accomplishing the increased revenues is in increasing the performance in our capabilities and ensuring that our metering devices are operating accurately through proper installation, maintenance and surveillance.

WATERLINE FREEZE PROTECTION AND AQUA-FLO

by

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INTRODUCTION

Prairie residents may well have a keener appreciation of water than the average North American after all, as long as tales of the "Dirty Thirties" live on, and as long as we function in an agricultural based economy susceptable to the ravages of drought, water is then an understandable preoccupation.

It is probably this preoccupation that has caused an ever increasing number of residents to wonder in recent years about water related developments, and are cause for considerable concern. The matter of a pipeline to divert water to Regina and Moose Jaw; increasing consumption of water and maintaining water quality in the face of increasing amounts of effluent being released, are all concerns which weigh heavily on the minds of those concerned with water.

It would appear to me that the real problem is one of attitude. We are children of a culture born in a water rich environment. We have never really learned how important water is; we understand it, but we do not respect it. We have learned how to manipulate it, bu we have never really learned how to handle it. Where it is cheap and plentiful, we have totally ignored it, but in our Northern communities, where it has been rare and precious, we have spent it with shameful and unbecoming haste. We have moved it from place to place across the landscape with reckless abandon. We have pried open great natural storehouses of it and spent millions of years of accumulation in a few short decades. Everywhere we have poured filth into it. We have done all these things purposefully, and in the long run, that is all that really counts.

Water is something we have taken for granted, like breathing in and out, or watching the sun rise and set. But we cannot afford to take it for granted much longer.

For many, many years, knowingly or unknowingly, we have been deferring the cost of our water use practises into the future. Now, the future has arrived and the bill is about to come due.

For despite all we can do with water, despite all the pipes and pumps with which to move it and the chemicals with which to sterilize it and the filter with which to filter it and all the various and sundry hoops we are able to make it jump through and admire its fountains, there is one thing we cannot do; we cannot make it. Water is limited. Today's shortage will simply grow worse tomorrow, and all the money, all the engineering skill and all the laws in the world will not be able to prevent the fact that our children, and their children and their children after them will be going to bed thirsty.

One of the most wasteful and blatant misuse of water occurs in the freeze protection of our service lines and mains.

Water is free and abundant, but transporting it from its source to the user is very expensive. Indiscriminate wastage is making our water precious!

Ever since man began building water and sewer systems in the regions of our planet that are subjected to long and severe winter, there have been the problems as to how to keep these systems from freezing. Of major concern is the smaller diameter pipes which have less of a specific and latent heat available. The design freezing times for water service lines usually takes only a few hours.

Since the freezing is caused by the penetration of frost down through the soil to the pipes, the general practice in the northern communities was to bury the lines below the frost line. This is not as practical and easy in the more northernly areas because of permafrost or because frost penetrates down to 10 feet (3 meters) and more, or because of a high water table or because of bedrock. Common practice in these areas is to insulate the lines, to install some form of freeze protection and to bury the service at a shallow depth.

A variety of insulation techniques and/or freeze protection systems are available and are used with varying degrees of success and expense. Unfortunately, no single or combined methods proved to be completely reliable and inexpensive to this point in time.

FREEZE PROTECTION

Domestic water temperatures range from 48°F in the summer to as low as 32°F in the winter, depending on the severity of the winter, the degree of snow cover the town has and the amount they heat their water in an attempt to prevent the water mains from freezing. Many Public Works Departments constantly monitor main temperatures in their areas, in an attempt to determine at what point the boilers should be activated.

While the raising of the water temperatures also serves to protect the service lines from freezing, in a case where demands are low, even the additional heat added at the pumphouse is not enough to prevent these lines from freezing.

In a number of communities, the method of waterline freeze protection is the discharge of large quantities of potable water directly into sewers. This practice is known as water bleeding or water wastage and is necessary, through design, to maintain flows for heat loss replacement.

Installation of water bleeders occurred at a time when municipal water supplies were abundant and relatively inexpensive to develop. However, due to the growth of communities and expansion of water distribution networks, many of these early systems have grown into substantial consumers of domestic water. Along with increased water consumption is increased concern over the large, dilute waste water discharges. Not only does bleeding pose a difficult technical task with respect to the treatment of cold, dilute waste water, but also the cost of this practice is, and is becoming, more expensive because of higher water cost. Of particular concern are the energy costs required in pumping water and waste water from one place to another. In communities which require the pretempering of water, these energy costs are also drained with the bleeder systems.

BLEEDING

There are two common types of direct bleeder discharges: water main bleeding and service line bleeding. On deadend water mains, stagnant water is prevented from freezing by installing large bleeders. Some of these bleeders are being eliminated through looping of the water mains to facilitate continuous system flows. However, in some cases, the requirement for water main bleeding is to keep low flow sections of sewer lines open. For service lines, continuous water movement to the sewer is provided through a small discharge line which taps off the household water service connection. In addition, indiscriminate bleeding often occurs through one or more faucet taps which run partially or fully open during the winter months. The elimination or control of service line bleeders would appear to be an expensive undertaking. However, if nothing is done about this problem, water and sewer costs, both capital and operating, will continue to escalate.

The following statistics may be of some interest. Smith, 1979, determined that Juneau, Alaska has a consumption rate of 1500 litres/person/day. Stanley Associates Engineering Ltd., 1974, determined that Whitehorse, Yukon has a consumption rate of 1680 litres/person/day. Jasper, Alberta has a consumption rate of 455 litres/person/day. This compares to a normal or average annual rate of some 450 litres/person/day.

HEAT TRACING

Installation of a heating cable in the service line has been suggested as a possible solution for the complete elimination of water bleeding. The principle of this alternative is that a heating cable would replace heat losses and maintain a minimum temperature for freeze prevention. The advantages of in-line electrical heat tracing is that there is maximum heat transfer efficiency and for this particular application, it is the only heat tracing system which could have serviceability with respect to an existing service line. With other heat tracing methods expensive excavation would have to be undertaken at which time alternate or additional freeze prevention measures would also become viable. However, it should be mentioned that with any of the electrical heat tracing systems, because of the relatively high cost of electrical energy, these systems are usually installed for freeze prevention in the event of operating upset, such as a prolonged no-flow condition, rather than as the primary method of maintaining a minimum operating temperature or to heat up the fluid.

The major deterrent to this solution for complete elimination of water bleeding is that the coating and joints of only a few cables, such as mineral insulant resistance cable, are approved for installation within pipes or for submerged conditions. In addition, resistance-type cables and wires available for installation within pipes specify that the return loop be formed outside the line. This requires a special fitting for the loop and return wire at the service line to main connection, which is difficult and expensive to install on existing pipe. For zone-parallel and self-limiting continuous parallel heating cables a loop is not necessary, however, there is not an approved water submersion cap and the end may also have to be brought outside the pipe. There may also be problems associated with the size of the cable and service line in which they are to be installed. On a prefabricated resistance-type in-line heat traced pipe the smallest line supplied is a 2.5 mm line. Other problems associated with this method of freeze protection are that the valves and other fittings must be by-passed; the cable must be removed to clean the pipes when some types of pipe repairs are necessary and an alternate freeze prevention system should be provided in the event of a power failure. Given these technical and installation difficulties, in-line heat tracing is not considered a practical solution for the elimination of water bleeding.

RECIRCULATION

This system entails the installation of 3/4" supply service line and a 1/2" return line connected to a small circulating pump that operates continuously. The circulation pump keeps the water in constant motion between the main and the house, thus keeping the lines from freezing.

The main advantage of a circulating system is that the pumps may be shut off in the summer, in order to conserve energy. As well, there is no wastage of water.

Freeze ups with this type of system have occurred for the following reasons:

1. Power outages
2. Mechanical failure
3. Low water temperature in low consumption areas

Examples of typical mechanical failures range from motor failure, coupling slippage or breakage to air locking and bearing failures.

Since there is no back-up system once the pump is not working, for whatever reason, the water line freezes if the problem is not detected. This is generally the case until the taps do not supply water.

AQUA-FLO

Operating Principle

Aqua-flo is a system and method of preventing water lines from freezing economically and with assured operation. Despite the problem and cost involved in the continuous bleeding of water through each water service line, it is a proven and indisputable fact that flowing or moving water does not freeze. Aqua-flo consists of a device to control the continuous flow of water 365 days of the year and 24 hours of the day. This is accomplished by a BLEED, CAPTURE and RETURN method. Water is constantly flowing but not from the tap into the sewer system. Water is bled from its source [usually the watermain] into a sealed container and when the container is full, it is pumped back to its source. This cycle is repeated over and over in a closed, sterile system once every hour.

Simply stated, this amazing invention maintains a constant flow of water through a single service line and returns the water into the system through the same water line without any loss. During periods of electrical or mechanical failure, flow is still maintained by overflowing from the holding tank into a sump or drain. This is Aqua-flo's back-up system, something that no other freeze protection system has.

This patented appliance consists of a 17 x 17 x 17" white polyethylene tank which has a 14.0 imperial gallon capacity, and orifice, liquid level monitoring device, electric motor, Procon rotary vane pump, by-pass valve, check valve, standard tie-in adaptors and tees, tubing, power control switch and cord, a base, pump and motor mounts. Any fittings, tubing, pumps, etc., that come in contact with water are either polyethylene or brass construction. The unit comes completely assembled and tested. The motor uses approximately 30 kilowatt hours per month.

How it works

To illustrate a situation of application of the system, the following describes a normal municipal water hook-up as could be found in a typical area. The operation of the appliance would be as follows:

> Water is bled from the normal house or municipal supply through an orifice into the holding tank. The water level in the tank increases and the trapped air is released through an air release valve. At a predetermined tank level, a pneumatic switch is tripped which starts the pump. The pump forces the water through a check valve into the stream ahead of the orifice and then back into the service line and water main. The orifice bleeds the water into the tank at a rate of one litre per minute. The filling time is approximately 40 minutes and the pump down time is about 2 minutes. One cycle lasts about 45 minutes. Variations will occur depending on the pressure at the water main. In case of electrical or mechanical failure, the water would continue to flow at the metered rate and would be discharged through a one-way valve into the drain and into the sewer main or sump. The flow rate remains unchanged. It never increases beyond the metered flow rate of 1 litre per minute. When the power comes back on, or the malfunction is rectified, the pump resumes its normal pumping cycle.

We are so confident of the effectiveness of our Aqua-flow unit, that we guarantee continuous freeze protection in the event of an electrical or mechanical malfunction. If it can be rpoven that your waterline froze because of our system malfunction we will pay the cost of having your waterline thawed out.

Why Aqua-flo?

Why do you need an Aqua-flo freeze protection system in your home? It will assure you of an uninterrupted supply of water along with many other benefits. Here are some of them for the homeowner.

1. Positive protection against freeze-up without a heavy burden on municipal systems.

2. Can be installed by a handyman.

3. Can be installed in any house, old or new.

4. Easy indoor installation of this new energy saving appliance will save you money and aggravation with water line freeze up.

5. No costly excavation required.

6. Low initial cost.

7. Costs less than heat tracing installations for new housing.

8. Low operating cost; intermittent operation.

9. No costly power bills as with heat tape.

10. No wasted water.

11. All parts C.S.A. approved.

12. All components guaranteed.

13. Extended pump and motor life due to low operation time.

14. Maintains protection during times of electrical or mechanical failure.

15. It has a built in back-up system, namely temporary bleeding.

16. No worries with power failure.

17. Heat absorbed in the house is returned to the municipal system saving municipal heating costs.

18. Available in industrial size.

19. Compatible with deep wells or pressure systems.

20. Ideal for construction camps.

21. Can be a standard "built in" system on all new mobile homes and recreational vehicles.

Aqua-flow has been developed in answer to the desperate need by the homeowner for an economical after market or retrofit failsafe water service line freeze protection.

Homeowners are finding the costs of operating their homes very high. Continued use of electricity to heat water lines is very expensive and impractical and the dual water circulating pump system is both expensive to operate and maintenance costs are high. Combined with the many elements of potential failure, incidence of lines freezing up and bursting, the attending costs of calling our service personnel if and when available, renders conventional methods and systems inoperable and very expensive.

For a Municipal Authority

In addition to the benefits a homeowner receives from Aqua-flo, there are perhaps greater benefits to be realized by a village, town or city and its tax paying residents.

1. Greater water consumption often requires expanded headworks, larger reservoirs and larger supply mains to supply an ever increasing demand for water. Indiscriminate bleeding places a burden on sewer treatment facilities particularly during the winter months. A water

and sewer system designed with a "bleed and return" system as Aqua-flo could eliminate costly water and sewer upgrading projects.

2. As the water bleeds into a holding tank in a warm home, it absorbs heat and is returned to the water main. This would help reduce municipal heating costs where water must be heated before distribution.

3. A significantly lesser amount of cold diluted waste water needs to be treated.* There is less danger of frozen sewer lines as well.

4. Aqua-flo is the failsafe answer to water conservation and resultant energy and cost saving.

* It is easier and cheaper to treat normal strength warmer sewage than the colder, dilute sewage.

Facts and Figures

The indisputable conclusion reached by various studies and research is that bleeding is the one and only way to failsafe freeze protection when it comes to water service lines. There is, however, bleeding with a difference. The Aqua-flow system is one thing but indiscriminate and uncontrolled bleeding is another. Most systems are designed to accommodate and approximate 450 litres/day rate of consumption. What happens when this rate is exceeded?

One bleeder, at the recommended rate of .25 gallons per minute will bleed off and waste 15 gallons of water in one hour or 360 gallons in one day or 10,800 gallons in a month. How many homeowners who have bleeders meter the flow to exactly the recommended rate? Generally the rate is 3 to 4 times the recommended rate. At 4 times the recommended rate, one bleeder in one home could have wasted 43,200 gallons a month. Multiply these figures now by the number of bleeders a water utility may have. The wastage becomes astronomical and truly unbelievable.

The City of Saskatoon, Saskatchewan charges a rate of $0.01/cu.ft. for water and $0.006 cu.ft. of water used as a sewer charge. Basically, each cubic foot of water in Saskatoon costs a user $0.016/cu.ft. At this rate one bleeder in Saskatoon costs the homeowner and the City $27.69 a month. This is the cost when the bleeder bleeds at the recommended rate of .25 gallons per minute. Let us assume that bleeding continues for seven months of the year, this bleeder now costs somebody $193.54 a year. Double this rate of .25 gallons to 0.5 gallons per minute, and your cost is up to $387.00 a year. If the rate is 1 gallon per minute, this one bleeder becomes rather expensive at $744.00 a year. This is more than the price of one Aqua-flo appliance.

Now if we carry this one step forward and multiply the number of bleeders the City of Saskatoon could have, we can get an idea of what the true costs of supplying water is to all those that bleed their lines for freeze protection. One bleeder cost the City $27.69 a month. Ten bleeders west $276.90 a month, one hundred bleeders $2,769.00. Do they bleed 1 gallon a minute? Then the cost is $110.77 for one, $1,107.70 for ten or $11,077.00 for one hundred bleeders.

Let us assume that a certain town has four hundred bleeders in operation (some have many more). Let us also assume the City of Saskatoon Water and Sewage charges a bleeder rate of 0.50 gallons per minute. These four hundred bleeders cost the town $154,828.00 for seven months of bleeding. At this rate, the town would recover the costs of installing an Aqua-flo appliance for each bleeder in less than two years.

CONCLUSIONS

We have talked about three main systems of freeze protection and their advantages and inherent drawbacks. The conclusions reached in various studies done on freeze protection are generally the same, namely that bleeding is the surest and simplest way, albeit the most costly in the long run. The general recommendations point towards some method of controlling indiscriminate bleeding or year round bleeding.

Some have suggested installing water meters to control this problem by forcing homeowners to pay for excessive water wastage. Another idea was to levy fines for bleeding lines during frost free times. This would entail a policing system, spot checks or routine systematic checking of every home for bleeders during the summer and excessive flow rates during the winter season.

It would appear that Aqua-flo, a failsafe, energy saving appliance that incorporates proven principles that ensure the prevention of water service lines from freezing up in the cold weather in houses, house trailers, cottages and temporary work camps, is the obvious answer to the problem of water service line freeze protection.

ASSESSING TRACE LEVEL IMPINGEMENTS AT SURFACE SUPPLY RESERVOIRS

by

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INTRODUCTION

The City of Calgary obtains water for potable water production from the watersheds of the Elbow and Bow rivers (Figure 1). Although both river systems rise in the Rocky Mountains and share many other similarities, there are significant differences that affect the susceptibility of each source to deterioration in quality through trace contamination. Firstly, the Elbow River system has only one tenth the average flow of the adjacent Bow River system at Calgary (Table 1). Also, its terminal reservoir, the Glenmore Reservoir, undergoes considerably less flushing than the corresponding Bearspaw Reservoir on the Bow River. Secondly, this situation is aggravated by the location of Glenmore Reservoir almost entirely within city limits. Thus, the Glenmore Reservoir receives urban runoff from storm events and is used for limited recreational activity. It is easy to see why concern in recent years over source water deterioration in Calgary has focused primarily on Glenmore Reservoir and the Elbow River.

FIGURE 1

Surface Water Sources for the City of Calgary

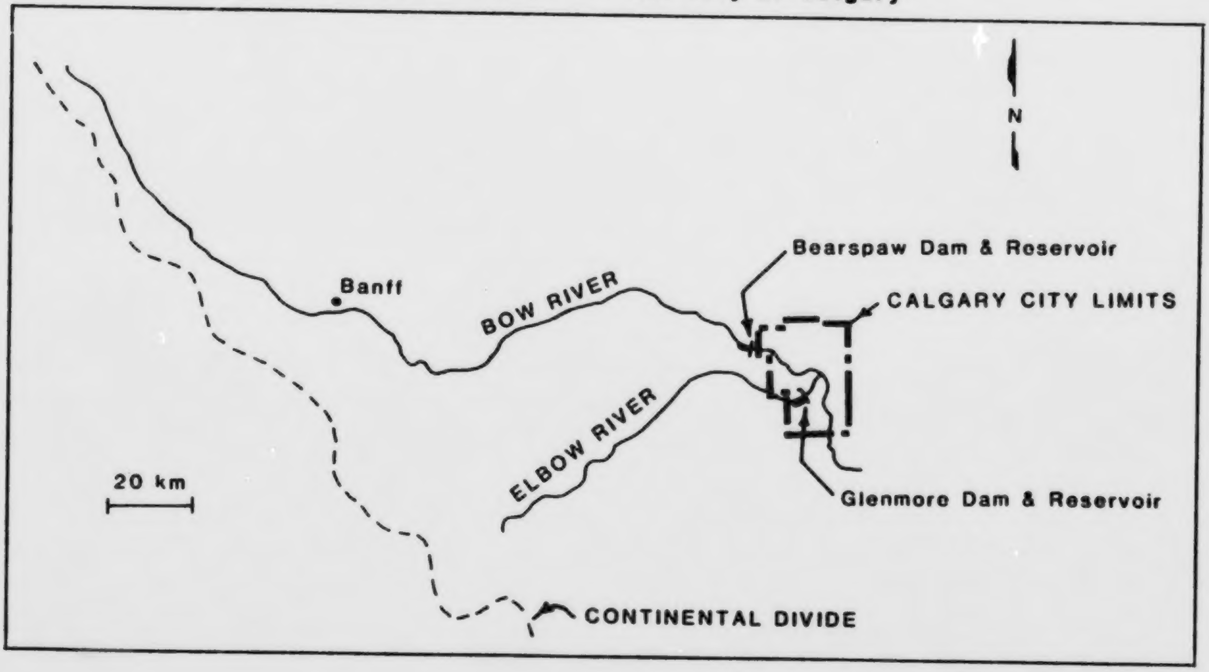


TABLE 1

Comparison of Calgary's Two Surface Water Supplies

	Elbow River Watershed and Glenmore Reservoir	Bow River Watershed and Bearspaw Reservoir
Watershed Size	1220 km^2	7860 km^2
Annual Yield	2.9×10^5 dam^3	2.9×10^6 dam^3
Reservoir Capacity	2.8×10^4 dam^3	3.0×10^3 dam^3
Reservoir Retention	26 days	0.4 days

Water quality surveys have been conducted in both watersheds though most effort has been expended in the Elbow River system. The City of Calgary has undertaken three separate studies in this regard. The first study, the Elbow River Watershed Study (1), carried out sampling of the Glenmore Reservoir and storm sewers for an extensive range of water quality constituents in 1982 and 1983. Some 75 parameters were analyzed at approximately 15 sites over the two year period. The second study, the 1984 Glenmore Reservoir Summer Water Quality Study (2), undertook more intensive sampling for a more selected group of parameters. Approximately 5,600 tests for 20 parameters were run over a four month period with particular emphasis on documenting impacts of recreational use and storm events. The third study (3) conducted a comparative water quality sampling program in the Bearspaw and Glenmore Reservoirs. Water sampling networks were established to specifically compare both reservoirs. During a one year sampling period, variations in selected microbiological, chemical and physical water quality parameters from spring runoff, reservoir turnover and through early winter at several locations in the Glenmore and Bearspaw Reservoirs were compared.

These three studies have generated an abundance of diverse information that will be used in future watershed management programs. This report only highlights some of the findings as they pertain to the subject of trace contaminant impingements. The discussion follows three main categories, namely: enrichment, trace chemicals, and bacteria. Indications of storm water loading and effects of recreational activity on microbiological and chemical water quality in the Glenmore Reservoir are considered.

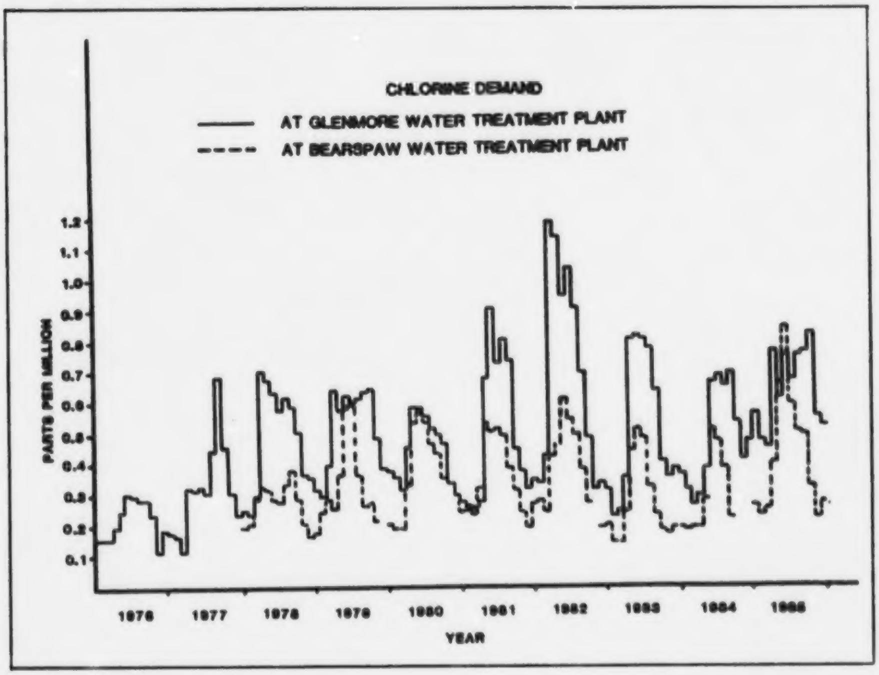
ENRICHMENT

Over the past few years, some Calgary water consumers have noticed an apparent increase in taste and odor properties. While such perceptions may be symptoms of an altering trophic status of the reservoir, they are certainly not considered hard evidence. It would be more valuable to quantitatively know how much trophic alteration is occurring and the tolerable maximum level of nutrient loading. In order to track highly variable and major processes such as eutrophication, long term water quality data can be very helpful. Two parameters which have provided such tracking capabilities at Glenmore Reservoir are chlorine demand and silica. Both parameters are directly related to autotrophic activity. Increases in autotrophic activity will increase chlorine demand through higher organic loading in the treatment process and will cause a depletion of silica through utilization by siliceous algae.

Chlorine demand is easily computed from accurate chlorine records (4). These calculations based on amperometric titration are available over the last ten year period at the Glenmore Water Treatment Plant and almost as long at the Bearspaw Plant (Figure 2). These records show that chlorine demand has increased approximately 0.05 ppm per year at the Glenmore Water Treatment Plant whereas no such trend exists at the Bearspaw Water Treatment Plant. Also, the Glenmore Plant requires per amount treated, about 50% more oxidant than Bearspaw. While these requirements are not excessive (the Glenmore Water Treatment Plant still needs only half the North American average oxidant), the long term increase only at the Glenmore Plant indicates increased nutrient loading into the Elbow River system.

FIGURE 2

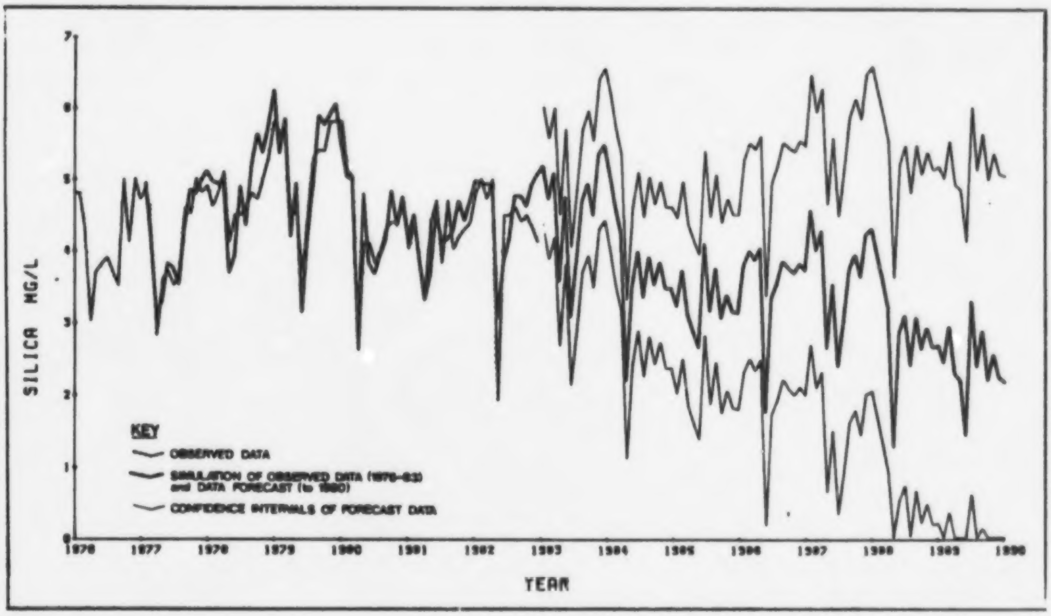
Chlorine Demand at Calgary's Water Treatment Plants



Monthly silica measurements in raw water provided to the Glenmore Water Treatment Plant have been routinely carried out. A Box Jenkins analysis of seven years of monthly silica data (1976 to 1983) is shown in Figure 3. This analysis projected a decrease in silica levels beginning in 1983. Actual silica levels recorded since 1983, though not identical to Box Jenkins projected values, confirm the anticipated silica depletion trend predicted by the model. Thus, both chlorine demand and silica level trends show increased nutrient loading to the Glenmore Reservoir and indicate a need to define the assimilative capacity toward any further increases.

FIGURE 3

Time Series Analysis of Glenmore Reservoir Raw Water: Silica



One indication of the extent of nutrient assimilation which has already occurred is the Vollenweider relationship [5] which compares phosphorus loading with hydraulic loading. This relationship categorizes Glenmore Reservoir as bordering between eutrophy and mesotrophy (Figure 4). Another accepted indication of trophic status is the level of chlorophyll 'a'. In the summer of 1984, 421 measurements of chlorophyll 'a' were carried out in the Glenmore Reservoir. The mean chlorophyll 'a' concentration at various points in the reservoir ranged between 0.88 and 1.78 ug/L with the maximum observed value of 6.06 ug/L. Such a regime would classify Glenmore Reservoir as oligotrophic or oligo-mesotrophic (6). Because trophic categories can be thought of in terms of probability distributions rather than in terms of clear cut distinctions, the two indicators may be resolved by considering the reservoir as mesotrophic. Thus, it appears unwise to presume that an additional assimilative capacity exists which would allow for additional nutrient input without substantial deterioration in water quality.

TRACE CHEMICALS

While gross chemical contamination of many North American and European drinking water sources has captured considerable attention, the lack of major industrial activity in Calgary's watersheds would preclude that type of contamination upstream of Calgary. However, low level contaminants from urban catchment runoff or from agricultural practices can be introduced into the watershed. Table 2 illustrates the intermittent nature of contamination from urban runoff. Such a situation poses special monitoring requirements because resultant concentrations in the reservoir are very low and loads are not consistently delivered.

FIGURE 4

Comparison of the Annual Areal Hydraulic Phosphate Loading Relationship for Various Lakes.
Phosphate Data Used for Glenmore Reservoir Calculation was taken from 1982 Spring Turnover

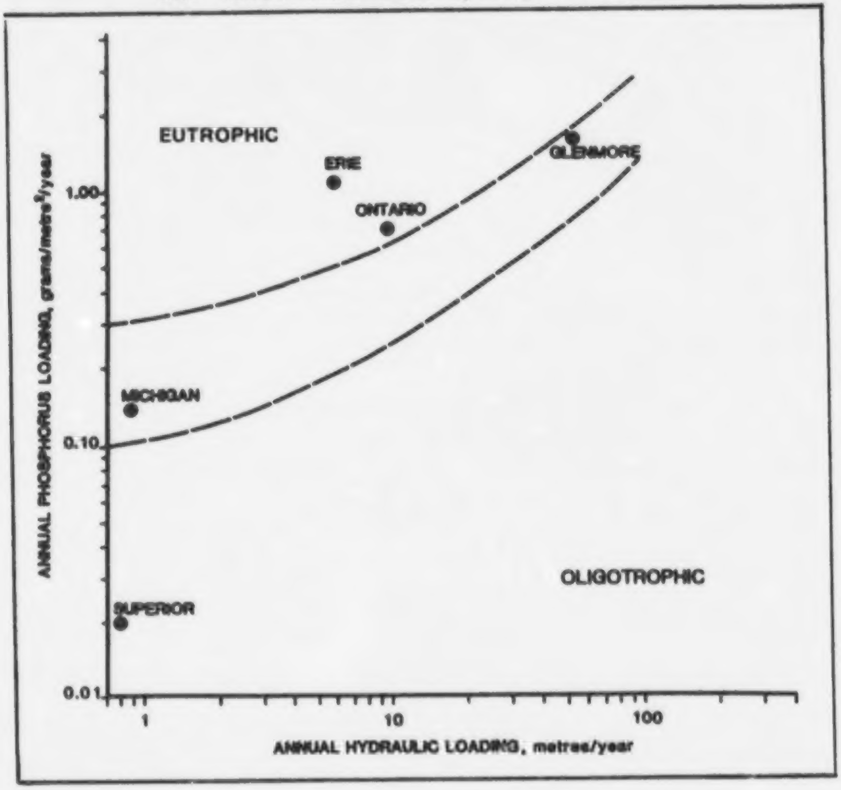


TABLE 2

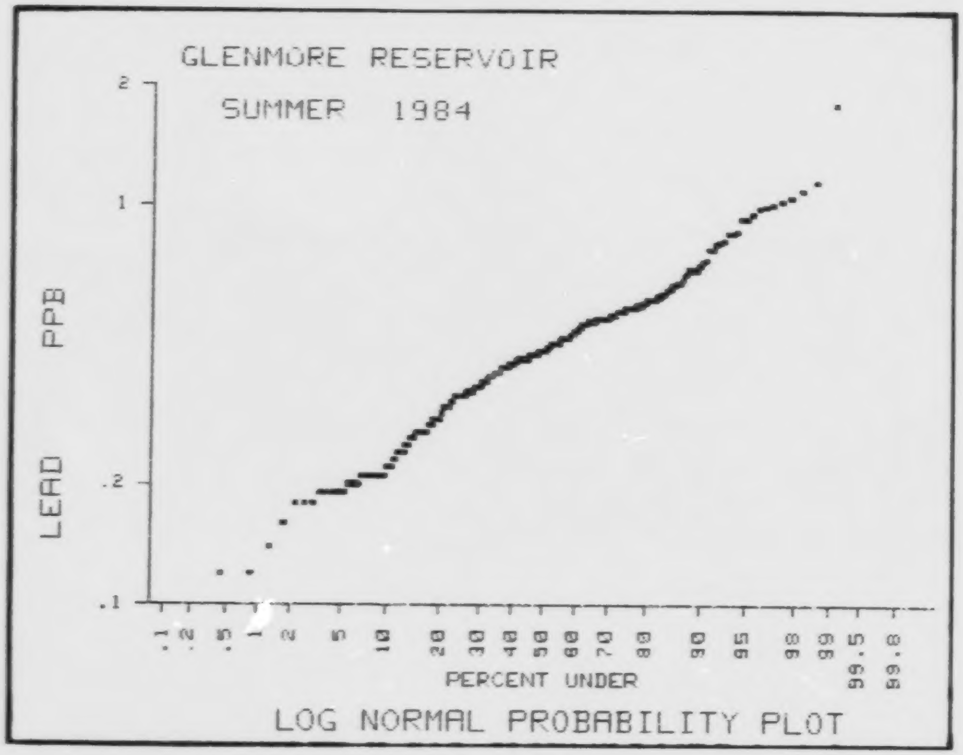
Synthetic Organics Detection in Stormwater During Runoff Events
(1982 - 1983)

Parameters	Number of Events Samples	Number of Events Containing Contaminants
Hydrocarbons	6	6
Phenolics	6	6
Naphthalene	6	1
a-BHC	6	1
Lindane	6	2
DDT	6	1
PCB	6	1
Dicamba	6	2
2,4 - D	6	4
2,4,5 - T	6	2
MCPA	6	2

One contaminant of particular concern is lead. Elevated lead levels are found in the atmosphere over all cities due to automobile emissions. Atmospheric lead typically causes concomitantly elevated lead levels in runoff water from urban catchments. Storm water modelling studies suggest that such urban runoff may supply 50 - 60% of total lead load into the Elbow watershed. An intensive survey of lead levels throughout Glenmore Reservoir was conducted in 1984 (Figure 5) (2). During that survey the lead regime in Glenmore Reservoir complied 96% of the time with the Federal ultimate water quality goal of 1 ppb (7). The maximum acceptable lead concentration for drinking water consumption is 50 ppb. These very low levels together with the generally linear log-normal relationship indicated a lack of pollution episodes during the sampling period.

FIGURE 5

Log-Normal Probability Plot of Lead Concentrations in Glenmore Reservoir



Synthetic organic compounds are also contaminants of concern because of their widespread use. However, since 1982 when trace monitoring for organics was initiated, the raw water to both plants contained less than detection limits of synthetic organics. In order to detect these compounds monitoring would have to take place closer to their sources. In the 1985-86 comparative study of the Glenmore and Bearspaw Reservoirs, water samples were collected for the analysis of three groups of specific organic compounds. Over 80 samples collected from five sites in the Glenmore Reservoir, two storm sewer outfalls to the Glenmore Reservoir and five sites in the Bearspaw Reservoir were analyzed for the phenols (8), base-neutral extractable compounds (9) and nonchlorinated volatile organic compounds [10] listed in Table 3. Samples from storm sewer outfalls invariably contained more compounds at higher concentrations than either Glenmore Reservoir or Bearspaw Reservoir. For example, storm sewer samples contained 25 to 50 ppb of the six hydrocarbon residues quantitated in this study. A wide variety of branched chain alkenes and unsaturated simple alkenes were also detected at similar concentration levels in these urban runoff effluents. Reservoir sites at Glenmore and Bearspaw contained 6 to 16 ppb and 5 to 25 ppb respectively, of the long chain alkane hydrocarbons. Interestingly, no significant $(p>0.05)$ difference between surface and bottom depths at the same reservoir site or between reservoir sites was observed. Average concentrations of aliphatic hydrocarbons in the Bearspaw Reservoir, however, were consistently greater than those detected in the Glenmore Reservoir.

TABLE 3

Organic Compounds Monitored (1985 - 1986) in the Glenmore Reservoir, Bearspaw Reservoir and Urban Storm Sewer Outfalls in the Glenmore Reservoir

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GROUP 1 - PHENOLS

Phenol
2-Chlorophenol
2,4-Dimethylphenol
4-Chloro-3-methylphenol
2,4-Dichlorophenol
2-Nitrophenol
2,4,6-Trichlorophenol
Pentachlorophenol

GROUP II - BASE-NEUTRAL COMPOUNDS

A) Pollutants and Aromatic Hydrocarbons

1,4-Dichlorobenzene
Bis (2-chloroisopropyl) ether
Hexachloroethane
Nitrobenzene
Dimethyl Phthalate
Acenaphthalene
Fluorene
4-Chlorophenyl phenyl ether
4-Bromophenyl phenyl ether
Anthracene
Fluoranthene
Butyl Benzyl Phthalate
Chrysene
Ethyl Hexyl Phthalate
Benzo (b) Fluoranthene
Benzo (a) Pyrene
Dibenzo (a,h) Anthracene
Benzo (g,h,i) Perylene

B) Long Chain Alkane Hydrocarbon

Dodecane
Tetradecane
Heptadecane
Eicosane
Tricosane
Hexacosane

NONCHLORINATED VOLATILE ORGANICS

Benzene
Toluene
Ethylbenzene
o - Xylene
m - Xylene
p - Xylene

A number of polyaromatic hydrocarbons (PAH's) included in the base-neutral compound group are formed as a result of incomplete combustion of organic compounds in the presence of insufficient oxygen (11). In water, PAH's are quite stable and remain there for long periods of time. Storm sewer outfalls to the Glenmore Reservoir contained 0.1 to 15 ppb of the polyaromatic compounds naphthalene, fluorene, anthracene, fluoranthene, benzo(B)fluoranthene and benzo(A)pyrene. A wide variety of xylene isomers, C3-benzenes (C_9H_{12}), C4-benzenes ($C_{10}H_{14}$), methyl-and dimethylnaphthalene isomers were also detected. Although all of the polyaromatic hydrocarbon residues identified in the urban runoff samples were detected in both reservoirs, levels never exceeded 0.1 ppb. Similarly, while storm sewer effluents contained up to 50 ppb of ethylhexyl phthalate and butylbenzyl phthalate these compounds were detected in reservoir samples at concentrations less than 5 ppb. None of the other pollutants listed in Group IIA in Table 3 were ever detected in storm sewer effluent or reservoir samples during the one year study period from May, 1985 to May, 1986.

Phenol, cresols and alkylphenols formed by decomposition of plant fulvic and humic substances are common constituents in surface waters. Some phenolic compounds are also components of industrial waste and sewage (12). Storm sewer effluents emptying into the Glenmore Reservoir contained total phenol concentrations ranging from <0.1 ppb to 15 ppb. Of the nine phenols listed in Table 3 only phenol, 2-nitrophenol, 4-nitrophenol and pentachlorophenol residues were detected. Phenol itself was always the major component present with values ranging from 0.15 ppb to 14 ppb. Pentachlorophenol, the only chlorinated phenol residue detected, was present at <0.1 ppb to 0.5 ppb in urban effluent samples. Storm sewer effluents also contained *o*-, *m*-, and *p*-cresol isomers, and a wide variety of methyl-, ethyl- and propylphenol isomer residues. The mean phenol concentration in the two reservoirs was 0.8 ppb, which compares favorably with the Federal guidelines ultimate objective (7) of less than 2 ppb. The urban storm outfalls do not appear to be contributing a significant load of phenolic compounds to the Glenmore Reservoir since similar concentration ranges were detected in both sample types.

The third group of organic compounds quantitated were the purgeable aromatics benzene, toluene, ethylbenzene and the xylenes. These compounds are petroleum and coal byproducts, and are also common constituents of gasoline (11). Recently (13), the United States Environmental Protection Agency proposed a maximum contaminant level of 5 ppb for benzene and 680 ppb for ethylbenzene in drinking water. The volatile nonchlorinated organics were detected at concentrations ranging from 0.1 to 0.2 ppb, with benzene and toluene being the major components in the reservoir samples. The xylene isomers were never detected in any Glenmore Reservoir or Bearspaw Reservoir sampling sites. In contrast all six volatile hydrocarbon residues were frequently detected in two storm sewer outfalls in the Glenmore Reservoir. Concentrations were highly variable with total values for the six volatile hydrocarbons ranging from below detection to over 10 ppb. In storm sewer samples benzene and toluene were also detected most frequently. The storm sewers may represent a major loading source of volatile hydrocarbon residues to the Glenmore Reservoir. Levels of these compounds were, however, always below 1 ppb at all sampling sites in the reservoirs.

BACTERIA

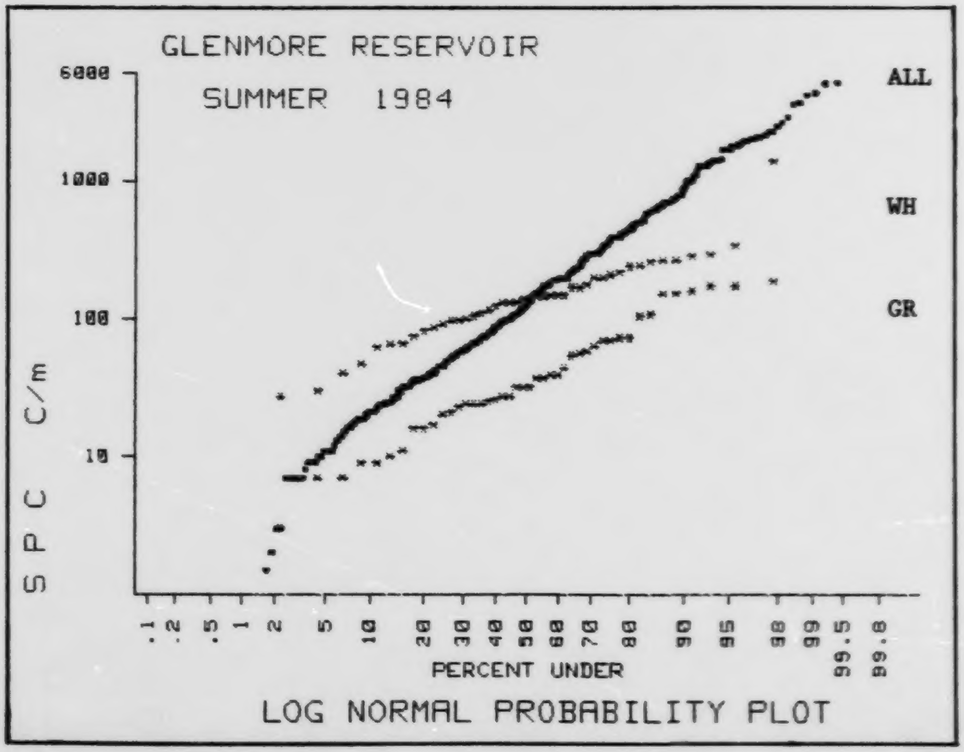
Bacteria comprise an important natural component of all aquatic ecosystems. They play a major role in the nutrient cycles, even competing with algae for phosphorus. While pristine waters have a small yet very diverse population of natural aquatic bacteria, more enriched waters have fewer species but higher populations. Water quality surveys traditionally include measurements of the densities of fecal coliforms, fecal streptococci, and heterotrophic bacteria.

The Standard Plate Count (SPC) was used in the 1984 Summer Study (2) to explore the flux of heterotrophic bacteria in Glenmore Reservoir. It is acknowledged that the SPC method underestimates the total heterotrophic population by two to three orders of magnitude. Nonetheless, its traditional use in evaluating the drinking water treatment process has led to a long term SPC record for source water samples at the Glenmore Water Treatment Plant intakes. In 1984, 561 samples were collected and analyzed for SPC (Figure 6). Three general conclusions were drawn from this survey: the SPC in the Elbow River was directly related to flow, SPC densities near the bottom of the reservoir were significantly higher ($p<0.05$) than in surface samples, and the SPC at the source water intakes was significantly lower ($p<0.05$) than the SPC of all other sites. All these conclusions are consistent with normally operating heterotrophic populations without significant disruption (pollution events).

FIGURE 6

Log-Normal Probability Plot of Standard Plate Count Values in Glenmore Reservoir

WH: Mouth of Elbow River into Reservoir (Weasel Head)
GR: Pump Station at Dam (Glenmore Raw)

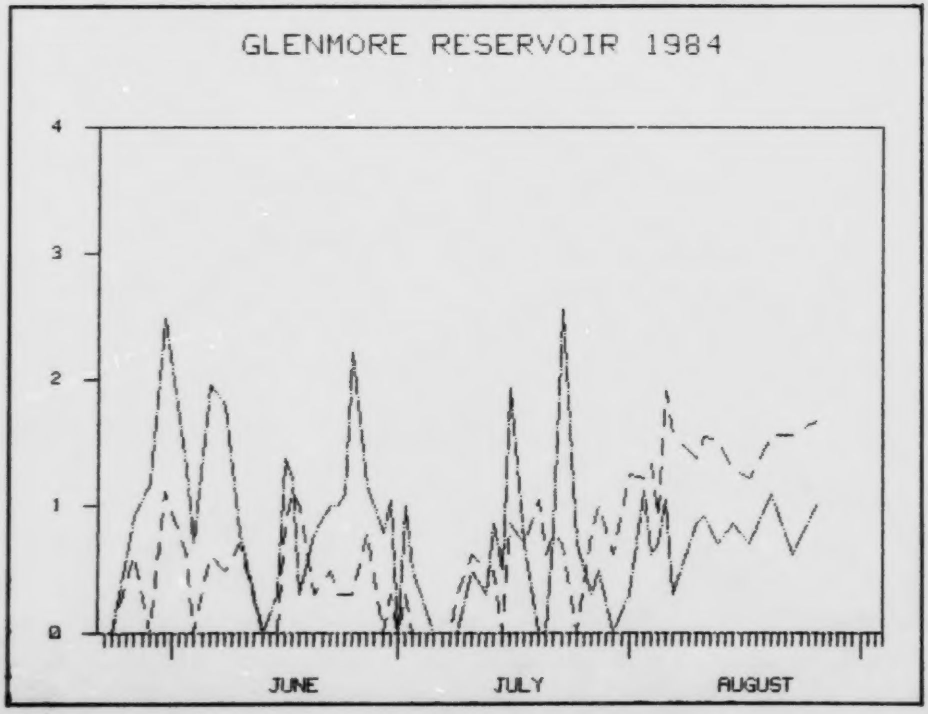


Fecal indicator bacteria were measured in all three studies. Over 700 samples have been collected and analyzed. All three studies indicated a relatively unpolluted aquatic system. The most comprehensive study was the 1984 Reservoir Study [2] where 590 samples were analyzed for fecal coliforms (FC) and 595 samples were analyzed for fecal streptococci (FS). While fecal coliforms ranged up to 249 per 100 mL, the arithmetic mean for all sites was 12 per 100 mL. FS ranged from less than 1 to 600 per 100 mL; the arithmetic mean for all sites was 18.3 per 100 mL. Three general conclusions were drawn from the results of the study. Firstly, FC and FS densities in the Elbow River were significantly higher ($p<0.05$) than densities at all reservoir sampling sites and were directly related to flow. Secondly, FC within the Heritage Cove area of Glenmore Reservoir were uniformly distributed both vertically and horizontally and were significantly higher ($p<0.05$) than other frequently sampled locations in the reservoir. Thirdly, FC densities at the source water intakes were consistently lower than all other sites.

The significant difference in Heritage Cove FC and FS results as compared to other reservoir sites warranted further investigation. As the 1984 summer sampling session progressed, marked changes in both the absolute and relative abundance of both fecal indicator bacteria were observed. Two distinct phases of FC/FS abundance were discerned, as shown in Figure 7. The early phase was characterized by erratic FC and FS densities. FS nearly always outnumbered FC during this phase. The later phase was characterized by FC densities exhibiting a clear and statistically significant ($p<0.05$) rise. The maximum FC values at all sites occurred during the later phase. In addition to the overall rise in FC, FC outnumbered FS in all samples collected after July 26, 1984.

FIGURE 7

Indicator Bacteria in Heritage Cove at Glenmore Reservoir.
Dashed Line: Fecal Streptococci
Solid Line: Fecal Coliforms
Ordinate: Log of Colonies per 100 mL



This late summer increase in FC and the unusual dominance of FC and FS in the reservoir at this time were attributed to an external or internal loading factor. Multiplication and/or increased survival capabilities of FC over FS is not a sufficient explanation for this late summer phenomenon. Neither FC nor FS are believed capable of multiplication in the environment [14] and, in fact, FC die off faster than FS [15]. This implies the existence of a loading source contributing a greater number of FC than FS.

Fecal pollution associated with recreational use of the reservoir could explain the increase in FC. However, these data do not unequivocally prove recreationally related contamination; such proof requires that the FS density exceed 100 per 100 mL which was not the case. Also, data from other years did not clearly confirm this trend. Though some scattered data in 1983 fell into the pattern displayed in Figure 7, the 1985 data were not consistent with the pattern. Persistent tracing of the peculiar levels of indicator bacteria in the late fall of 1985 led to the discovery of a sanitary cross connection in a storm sewer line. Input from that situation no doubt complicated the monitoring picture.

CONCLUSION

Trace impingements can be detected prior to eruption into major problems. Information from trace monitoring programs typically does not provide unequivocal evidence of pollution events, yet such information can be very useful. In the case of enrichment, though the Glenmore Reservoir is not eutrophic, it has been concluded that very little further assimilative capacity exists in Glenmore Reservoir without the danger of shifting into an undesirable trophic category. In the case of trace chemicals, though gross contamination is not occurring, detection of very low levels of metals and synthetic organic compounds calls for continual vigilance. In the case of bacteria, though fecal contamination through recreational use is not proven, the need for an on-going monitoring network is certainly indicated.

An overall assessment at this time shows that no one single impingement can be regarded as a dominant factor nor has significant deterioration yet been detected. Rather, even though continued monitoring is called for, the quality of both surface sources for water in Calgary remains highly acceptable for the conventional treatment received.

ACKNOWLEDGEMENTS

The 1984 Glenmore Reservoir Summer Water Quality Study was funded in part by a grant from Employment and Immigration Canada (Summer Canada Works). The Comparative Study of Glenmore and Bearspaw Reservoirs (1985-86) was supported financially by two Alberta Manpower programs: Environment Employment Program and Summer Temporary Employment Programs.

REFERENCES

(1) Marshall Macklin Monaghan Limited, "Elbow River Watershed Study" Report to the City of Calgary Engineering Department, 1985.

(2) Hargesheimer, E.E. and C.M. Lewis, "1984 Glenmore Reservoir Summer Water Quality Study", internal report, 113p, 1985.

(3) Hargesheimer, E.E. and C.M. Lewis, "Comparative Study of Glenmore and Bearspaw Reservoirs", internal report in preparation, 1986.

(4) Seidner, R.T. and E.E. Hargesheimer, "Chlorination of Calgary City Water: Quantifying Trends". Proceedings of the 35th Annual Western Canada Water and Sewage Conference, p 125-136, 1983.

(5) Vollenweider, R.A., "Input-Output Models with Special Reference to the Phosporous Loading Concept in Limnology". Schweizerische Zeitschrift fur Hydrologie, Vol. 37, 1975.

(6) Reckhow, K.H. and S.S. Chapra, "Engineering Approaches for Lake Management, Vol 1: Data Analysis and Empirical Modelling", 340p., 1983.

(7) Health and Welfare Canada, "Guidelines for Canadian Drinking Water Quality 1978", 78p., 1979.

(8) Coutts, R.T., E.E. Hargesheimer and F.M. Pasutto, "Gas Chromatographic Analysis of Trace Phenols by Direct Acetylation in Aqueous Solution", *J. Chromatog.* *179* (1979) 291-299.

(9) Hargesheimer, E.E., G.A. Irvine, A. Badakhshan and R.T. Seidner, "Pilot Studies into Effects of Disinfection Strategies on Drinking Water Quality". Proceedings of Second National Conference Drinking Water, April 7-8, 1986, Edmonton. In press.

(10) Purgeable Aromatics Methods 602, *Federal Register* Vol. 44, No. 233, December 3, 1979, p69473-69479. United States Government Printing Office, Superintendent of Documents, Washington, DC, 20402.

(11) Lappenbusch, W.L., *Contaminated Water and Your Health*, Lappenbusch Environmental Health Inc. (1986), Alexandria, Virginia, 22312.

(12) Buikema, A.L., Jr., M.J. McGinnis and J. Cairns, Jr., "Phenolics in Aquatic Ecosystems: A Selected Review of Recent Literature". *Mar. Environ. Res.* 2 (1979) 87 - 181.

(13) *Federal Register*, Vol. 50, No. 219 (11-13-85). United States Government Printing Office, Superintendent of Documents, Washington, DC, 20402.

(14) Clausen, E.M., B.L. Green and W. Litsky, "Fecal Streptococci: Indicators of Pollution". In: Headley, A.W. and B.J. Butka (eds.), *Bacterial Indicators/Health Hazards Associated with Water*. (1977) pp247-264. American Society for Testing and Materials, Philadelphia, P.A.

(15) Geldreich, E.E., L.C. Best, B.A. Kenner and D.J. van Donsel, "The Bacteriological Aspects of Stormwater Pollution". *J. WPCF* 40 (1968) 1861-1872.

POWER GENERATION FROM SLUDGE GAS AT THE BONNYBROOK SEWAGE TREATMENT PLANT

by

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SUMMARY

Considerable amounts of energy can be produced every year in a sewage treatment plant in the form of digester gas. Often this resource is largely wasted by flaring off the excess gas. This paper describes how this energy source has been utilized at the City of Calgary Bonnybrook Sewage Treatment Plant to generate electricity, shave peak electrical demand, improve power factor, and heat the buildings and process by burning digester gas in engine-generator sets and recovering the waste exhaust and engine heat. By using three modes of operation and sophisticated automatic controls, the power generation facility realizes substantial reductions in the cost of purchased electrical power, provides increased standby generating capacity in the case of utility failure, and improves overall plant energy efficiency.

HISTORY

Early in the pre-design phase of the Bonnybrook Sewage Treatment Plant (BBSTP), a study was undertaken to evaluate the alternatives for utilizing the sludge gas produced as a by-product of the digestion process. Among the alternatives were:

- Pipe the excess digester gas to other City of Calgary facilities elsewhere in the City, to be used to displace natural gas for heating.
- Sell the excess digester gas to nearby users of natural gas.
- Utilize the excess digester gas as fuel for City vehicles.
- Burn the digester gas in boilers to produce high pressure steam to run steam turbine generators to produce electrical power and heat.
- Utilize the digester gas as fuel for gas turbine-generators and recover the exhaust heat.
- Utilize the gas as fuel for reciprocating engine-generators and recover the exhaust and jacket water heat.
- Stay with the present practice of burning some of the digester gas in boilers to provide process and building heat, and flare off the excess gas.

The study was aimed at finding the most cost-effective and energy-efficient method of utilizing the digester gas, using currently available and proven technology, placing emphasis on:

- Economic viability considering capital costs, manpower, operating and maintenance costs, life cycle, and overall rate of return.

- Overall energy efficiency.

- Technical feasibility including proven performance under similar conditions elsewhere.

Some of the factors considered were:

- Projected quantity and variability of digester gas production.

- Marketability of the digester gas considering heat value and quality of the gas, and variability of plant requirements for and production of the gas.

- Cost of transporting the gas to other locations.

- Projected plant heating loads modelled to obtain heat demand as related to climatic data for Calgary.

- Projected plant electrical loads and peak demand.

- Integration with plant requirements for standby power.

- Electrical power output and heat recovered, related to gas production and plant electrical and heat requirements throughout the year.

- Feasibility of gas and heat storage.

- Electrical power generation for peak shaving as well as base load and the effect on plant utility bills.

- Projected increases in the value of electrical power generated.

After careful consideration, it was decided to proceed with a system based on burning the digester gas on-site in reciprocating engines, to generate electrical power and to recover the waste heat for uses at the plant.

Final design proceeded and the facility was completed in the fall of 1985 and has been operating successfully to date.

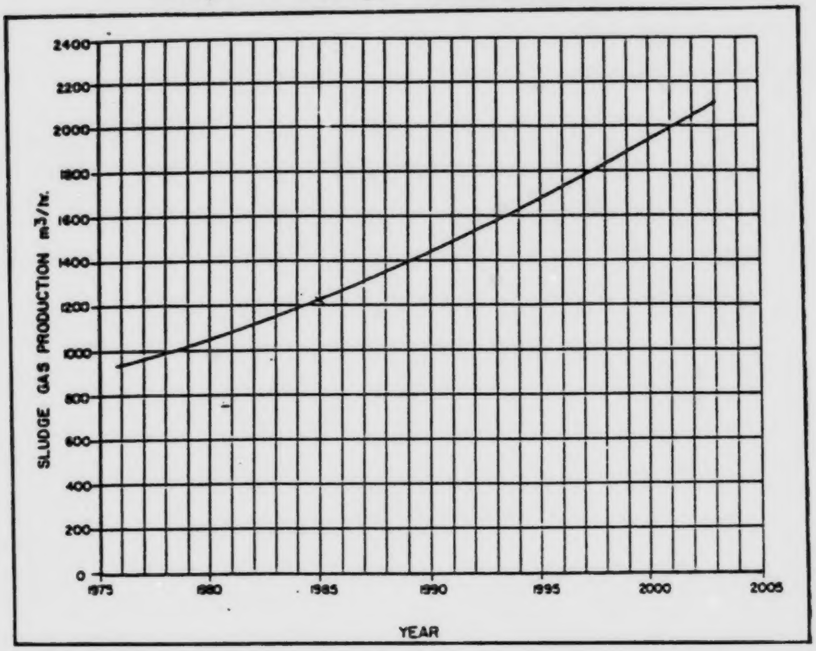
DIGESTER GAS PRODUCTION

Prior to the addition of the power generation plant, the digester gas was burned in boilers to heat the plant and the excess was flared off. Records show that in 1978, for example, 51% of the average 1000 m^3/hour of digester gas production was wasted by flaring.

With the expansion of the plant and the increased flows through the plant, increasing amounts of digester gas are produced every year. A projection is shown in Figure 1.

FIGURE 1

Projected Sludge Gas Production at Bonnybrook Sewage Treatment Plant



Analysis of the digester gas showed a methane content of 65.1%, and a gross heating value of 6.66 KWH/m^3 = (644 Btu/ft^3) which corresponds to about 65% of the heating value of pipeline natural gas.

The energy in the flared gas represents a valuable resource. Based on 1986 gas prices of $2.557 per gigajoule (after taxes and rebates), and compensating for the reduced heating value of digester gas, the replacement value of the energy in the sludge gas production in 1986 (estimated at 1300 m^3/hour) is $698,000.00.

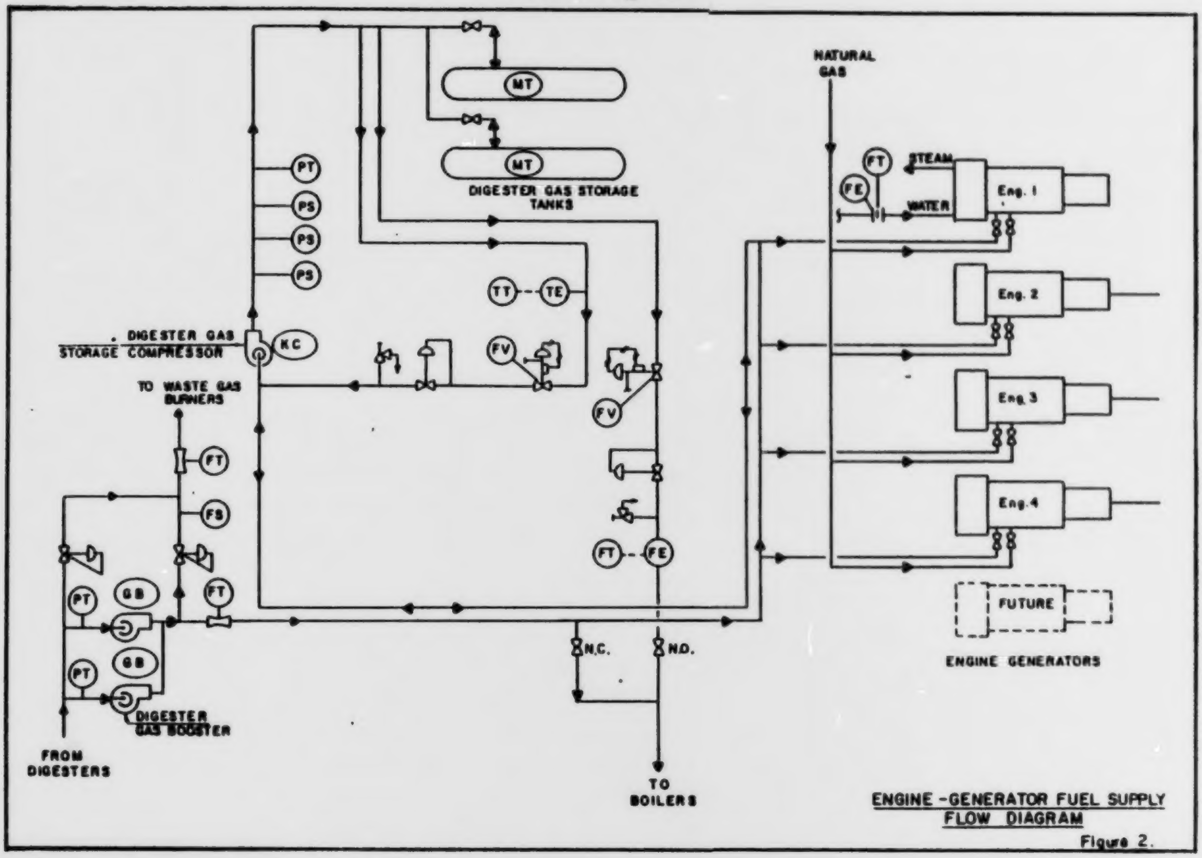
SYSTEM OVERVIEW

The power generation facility at Bonnybrook Sewage Treatment Plant consists of the following major components as shown in Figure 2.

- Four digester gas/natural gas fueled reciprocating internal combustion engines, each driving a generator set at 900 RPM. Each engine is equipped with a jacket water and exhaust heat recovery unit.
- Two digester gas booster compressors which transfer the sludge gas from the digesters to the power generation building. These compressors are variable speed, controlled to automatically maintain the flow to the engines equal to the rate of gas production in the digesters.
- One digester gas storage compressor and two cylindrical gas storage tanks designed to automatically store digester gas when the engines cannot utilize the available gas production.

FIGURE 2

Engine - Generator Fuel Supply Flow Diagram


NATURAL GAS
MT
MT
DIGESTER GAS STORAGE TANKS
PT
PS
PS
PS
TT
TE
DIGESTER GAS STORAGE COMPRESSOR
KC
FV
FV
TO WASTE GAS BURNERS
FT
FS
FT
FE
PT
GB
PT
GB
DIGESTER GAS BOOSTER
FROM DIGESTERS
N.C.
N.O.
TO BOILERS
FT
FE
STEAM
WATER
Eng. 1
Eng. 2
Eng. 3
Eng. 4
FUTURE
ENGINE GENERATORS

ENGINE-GENERATOR FUEL SUPPLY FLOW DIAGRAM

Figure 2.

Instrumentation and controls for the above are also connected to the computer data acquisition and control system (CDACS). This allows control loops to be monitored and adjusted from the local or remote operator terminals, and permits extensive display, trending and logging of process variables.

The engine generator units are housed in a building designed for flexibility, ease of operation and ease of maintenance. The building includes a second level control room overlooking the generator room, overhead travelling crane, mainfloor switchgear room, gas storage compressor room, space for a future engine-generator, high capacity ventilation system, and heat recovery/conversion systems. The building also features generous layout space for maintenance, truck access to the interior and extensive use of sound attenuation materials.

ENGINE-GENERATOR HEAT RECOVERY

The engines are naturally aspirated, V-12, 162 liter (9886 cubic inch) displacement, Cooper Superior Model 12G825.

Each engine is equipped with a dual fuel system to use either digester gas or natural gas. If the digester gas pressure falls below a preset level, the engines will automatically switch to natural gas without loss of power. Engine-generator output at 24°C ambient on digester gas is 780 KW. 850 KW can be generated using natural gas fuel.

Engine durability is enhanced by running the units at 900 rpm with a maximum output of 780 KW on digester gas, whereas the same engines, turbo-charged and natural gas fueled are rated at 1490 KW continuous. An automatic lube-oil makeup system with individual oil level regulators is provided. A pyrometer monitors temperature at each cylinder head and exhaust manifold.

The generators are rated 850 KW, 0.8 P.F., 4160 volt, 3 phase, 60 HZ and are equipped with brushless exciters. Each generator is capable of producing 645 KVAR to correct plant power factor. The generators are connected to the main bus using automatic synchronizers. Controls ensure automatic sharing of load and KVARS between operating units.

The heat recovery unit is vertical type Maxim Model TRP. Jacket water heat is recovered from the engine ebullient cooling system in the form of steam. Further heat is recovered from the exhaust gases which pass through the heat recovery unit producing steam. The steam is then passed through a steam/hot water converter. The hot water is used to heat the digesters and for building heat around the plant.

The engine-generators may be operated in any one of three modes: normal power generation, peak shaving, and standby power.

NORMAL POWER GENERATION

In this mode, the engine-generator's output is controlled by the fuel supply and the engines generally use all of the digester gas produced by automatically tracking the amount available. The generators are used to reduce the electrical demand and energy charges from the utility by generating while synchronized to the utility supply.

To match the engine's output to the varying sludge gas output of the digesters, the controls monitor the pressure in the main digester gas header and use a feedback control loop with an import/export controller and electronic load sharing and speed control governor. As main header pressure rises, because more digester gas is available, the controls increase the governor setting thus using more gas. As main header pressure falls, the governors are automatically set back to use less gas.

Most of the year, the plant operates in the normal power generation mode.

PEAK SHAVING

In this mode, the engine-generator's output is controlled by the operator. The engines do not track the digester gas supply, and any shortfall in digester gas is made up by natural gas.

The rate schedule for the plant used by the City of Calgary Electric System includes a factor "Peak Hour Demand" and a factor "Winter Peak Demand". A customer can realize substantial savings by reducing the plant electrical demand during these periods since the billing for every month of the year includes these factors.

To peak shave the plant demand, the operator sets all units to generate at maximum output while synchronized with the utility. The engines automatically switch to natural gas, one by one, until the digester gas supply is adequate for the number of units running on that fuel. Since the engines produce maximum power on natural gas, the operator may switch all units to run on natural gas to achieve maximum peak shaving.

STANDBY POWER

The generators provide standby power in the event the utility supply fails. In this mode, the engine-generator's output is determined by the load. The generators stop tracking the digester gas available and are automatically switched to natural gas to allow maximum power output.

When the utility fails with the generators on line, the plant is automatically disconnected from the utility and all nonessential loads over 7.5 KW are automatically switched to natural gas to allow maximum power output.

When the utility fails with the generators on line, the plant is automatically disconnected from the utility and all nonessential loads over 7.5 KW are automatically shed to prevent overloading the generators.

If the utility interruption is prolonged, the operators may put more generators on line (including two older 500 KW natural gas fueled standby generators) and restart some of the loads previously shed.

DIGESTER GAS STORAGE

Digester gas is stored by compressing the gas to a maximum of 1340 KPA (195 psig) and storing it in two large cylindrical tanks, each having a capacity of 2300 m^3 of gas at STP.

Whenever digester gas production exceeds the fuel consumption capability of the engines running, the excess initially goes up the flare stacks. The flow to the flare stacks is monitored and, when it exceeds a set point, the gas storage compressor starts and diverts the excess to the storage tanks.

During periods when more digester gas is needed by the engines (such as peak shaving), the controls can release the stored gas back into the main gas header to the engines. Stored gas can also be released to the boilers when additional heat is required.

POWER FACTOR CORRECTION

The generators are used to correct power factor in the plant. The benefits of using the generators for power factor correction instead of large banks of capacitors are:

- Control is variable and can thus be raised or lowered to accommodate varying plant loads.
- High inrush currents associated with switching large capacitor banks are avoided.

Power factor is the ratio of real power (KW) to apparent power (KVA). Certain loads, such as motors, draw both real power (KW) to drive the load, and reactive power (KVAR) for magnetizing current. The mathematical relationship is $KVA^2 = KW^2 + KVAR^2$.

Since the utility bills the plant on KVA demand, not KW demand, as well as energy used, substantial savings can be realized by improving the power factor as close to unity as possible. Improving the power factor does not change energy usage, but is accomplished by compensating for the reactive power (KVAR).

The generators are each capable of producing up to 645 KVAR of reactive power to correct power factor. Since the KVAR can be produced without changing the load on engines, the generators are equipped with automatic VAR/power factor controllers.

In the automatic VAR control mode, the generators will maintain whatever fixed KVAR output that has been set, regardless of varying kW output. The automatic VAR control mode is used at BBSTP to generate maximum KVAR during peak shaving periods; which results in maximum correction of power factor.

OTHER BENEFITS OF POWER GENERATION

Generating electrical power at Bonnybrook Sewage Treatment Plant benefits not only the plant but also the City of Calgary Electric System (CCES).

Since CCES purchases its electrical power from TransAlta Utilities, any reduction in system demand, both month to month and annually, results in substantial cost savings. Typically, peak system demand occurs in the winter months during a predictable time period each day. This results in the incentive rates for customers described under "Peak Shaving" as "Peak Hour Demand" and "Winter Peak Demand".

However, at other times of the year, CCES may wish to lower the system peak demand, and will contact BBSTP to bring more generators on line.

ECONOMICS

Using straightforward operating procedures, estimates show that, with 1985/86 electrical rates and present plant loads, the reduction in electrical power costs would amount to approximately $770,000.00 per year.

Calculations show that roughly 62% of this amount is due to reduction in peak demand (KVA) and the remaining 38% of the dollar amount is due to reduction in the energy (kilowatt hours) purchased.

The capital cost of the facility was approximately $8 million including engine-generator sets, building, mechanical systems, switchgear, control room, gas storage facilities and associated controls.

The economic benefits of the power generation will become even more important as electrical utility rates rise in the future.

PERFORMANCE EVALUATION
OF THE WASTEWATER TREATMENT PLANT
AT LAKE LOUISE

by

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INTRODUCTION

In the fall of 1983, a new wastewater treatment facility was commissioned by Parks Canada to treat the wastewater from Lake Louise Village and the nearby Chateau Lake Louise. The new facility replaced the wastewater lagoons that did not meet Environmental Protection Service (EPS) guidelines for municipal effluent discharges to surface waters. The treatment plant at Lake Louise uses the oxidation ditch variation of the activated sludge process with aerobic sludge stabilization.

The objective of this paper is to present a performance evaluation of the treatment plant based on the plant operation data for a 19-month period from June 1984 to December 1985. The data have been analyzed to determine the plant's effectiveness to meet the EPS guidelines. A statistical summary of plant operational parameters and their relationships is presented. The values of key parameters such as effluent soluble BOD and sludge production, as determined from the plant data, have been compared to the values calculated from theoretical design equations.

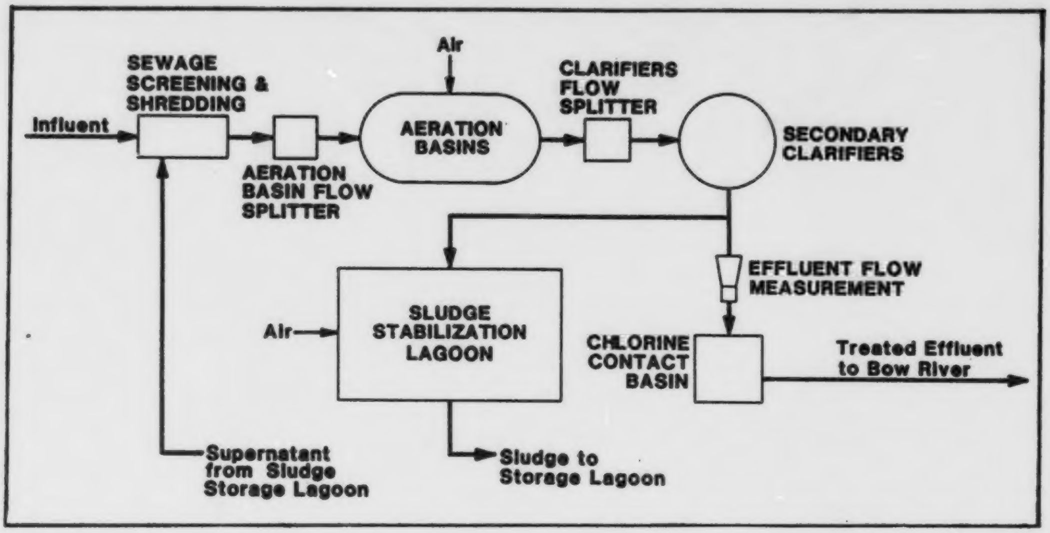
PLANT DESCRIPTION

Process Description

Figure 1 is a simplified process flow schematic of the Lake Louise Wastewater Treatment Plant. Wastewater from the Lake Louise area sewage collection system is pumped to a channel in the headworks area containing the screening and shredding equipment. In this area a coarse bar screen removes large suspended and floating matter and a sewage shredder chops the solids passing through the screens. The aeration basins splitter box, located downstream of the sewage shredder, splits flow equally between the two aeration basins. Each aeration basin is equipped with the jet aeration system to provide oxygen for micro-organisms for assimilation of organic food (BOD) and to keep micro-organisms in suspension. The mixed liquor from the aeration basins is divided at the clarifier flow splitting box and directed to the two clarifiers. In the clarifiers the micro-organisms settle, forming a sludge blanket in the bottom of the clarifiers. Most of the settled sludge is returned to the aeration basins and a portion is wasted to the sludge stabilization lagoon. The waste sludge is further aerated in the sludge stabilization lagoon to oxidize the residual organic matter in the waste sludge. The stabilized sludge flows to a sludge storage lagoon where the sludge is allowed to thicken by gravity. The supernatant from the sludge storage lagoon is pumped back to the headworks area.

FIGURE 1

Process Flow Schematic


Air
SEWAGE SCREENING & SHREDDING
Influent
AERATION BASINS
CLARIFIERS FLOW SPLITTER
SECONDARY CLARIFIERS
AERATION BASIN FLOW SPLITTER
EFFLUENT FLOW MEASUREMENT
Air
SLUDGE STABILIZATION LAGOON
CHLORINE CONTACT BASIN
Treated Effluent to Bow River
Supernatant from Sludge Storage Lagoon
Sludge to Storage Lagoon

The clear overflow from the clarifiers flows to the effluent flow measurement and chlorine contact basin. Here the effluent flow is measured and disinfected with chlorine solution. The chlorinated effluent flows to the Bow River via an outfall pipeline.

Waste Loads

The wastewater flow rates show substantial seasonal fluctuations due to service population changes from tourist to non-tourist season. As shown in Figure 2, maximum wastewater flows occur during the summer tourist season from mid-June to mid-August. Wastewater flows are also high during the winter skiing season from mid-December to mid-April. Lower flows occur during spring and fall when there are the least number of visitors in the area. In addition to seasonal fluctuations, wastewater flows can vary considerably between the weekdays and the weekends.

A summary of the plant design and actual waste loads is presented in Table 1. The waste loads for only the summer and winter seasons are given in the table because these seasons represent the maximum load conditions and are critical for the plant operation. The lower waste loads during spring and fall can usually be handled by using one of the two aeration basins.

As shown in the table, the actual plant flows experienced during 1985 were considerably higher than the design flow rates. The actual organic loads, however, were lower due to weaker than anticipated wastewater BOD and SS concentration.

FIGURE 2

Seasonal Flow and Temperature Variations

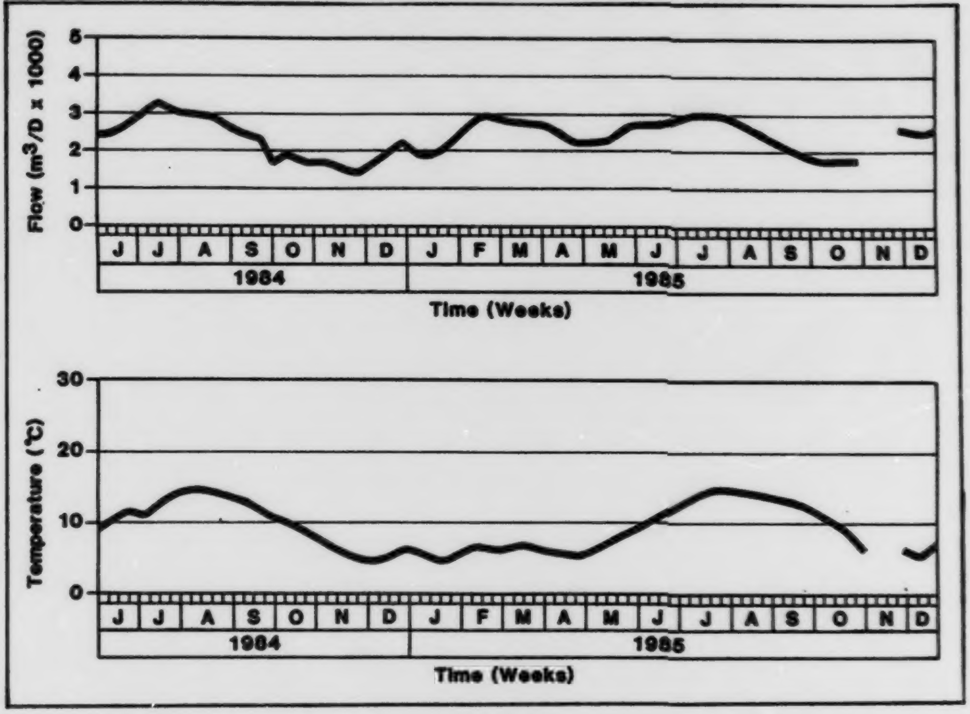


TABLE 1

Summary of Plant Waste Loads

Parameter	Design	Actual (1985)
Wastewater Flow		
. Summer Season - m^3/d	1700	2850
. Winter Season - m^3/d	1500	2480
Organic Loading		
. Summer Season		
- BOD$_5$ - kg/d	750	340
- SS - kg/d	850	460
. Winter Season		
- BOD$_5$ - kg/d	500	140
- SS - kg/d	510	180
Average Wastewater Temperature		
. Summer Season - ^{o}C	10	14
. Winter Season - ^{o}C	2.5	5.5

Note:

a) Summer season is from mid-June to mid-August

b) Winter season is from mid-December to mid-April

Wastewater Temperature

Wastewater temperatures are a significant consideration in the plant design as the temperature affects the rate of BOD assimilation in the aeration tanks and the rate of suspended solids settling in the clarifier. In general, colder wastewater takes longer to decompose and to settle out the suspended solids because of decreased metabolic activity. A 10°C drop in temperature lowers the metabolic activity by about one-half which has an effect on the process similar to reducing the number of organisms in the system. A literature review and evaluation of the extended aeration system prepared by EPS (1) have cited a reference indicating that the effectiveness of the settling tanks may be decreased by 20 to 40% for a temperature drop of 10 to 12°C. As shown in Figure 2, the wastewater temperatures at Lake Louise show sharp fluctuations between the summer and winter season. While the average temperature is about 14°C during the summer months, it drops to below 5°C during the winter months.

Special Design Features

The design of the Lake Louise plant posed three distinct problems.

1. Winter temperatures at the plant site can reach -40°C and the wastewater temperatures drop below 5°C.

2. Wastewater flow rates fluctuate sharply due to population changes from tourist to non-tourist season.

3. The site is within Banff National Park, necessitating minimum intrusion on the natural surroundings.

To meet the problem of sustained freezing air temperatures and cold wastewater, both the aeration basins and the clarifiers were covered to preserve heat. The process air is heated during winter months to maintain its temperature at about 0.5 to 1°C on the discharge side of the blowers. These design measures have been effective as the temperatures of mixed liquor in the aeration basins and the plant effluent have been about 1°C higher than the temperature of the influent wastewater.

To handle the fluctuating waste loads, the oxidation ditch variation of the activated sludge process was selected for its flexibility and ease of operation. Also, two aeration basins were provided to allow flexibility in handling the substantial seasonal waste load variation. While both basins are used during the summer and winter months, only one basin is usually required during the spring and fall. This also allows annual cleanup and maintenance of the basins without interrupting the normal treatment process.

To avoid clashing with the scenic landscape, a compact plant layout was developed and the colors of buidings were designed to blend with the surrounding forest lands. The compact layout also limited the exposure of plant operators to extreme cold weather conditions.

PLANT PERFORMANCE EVALUTION

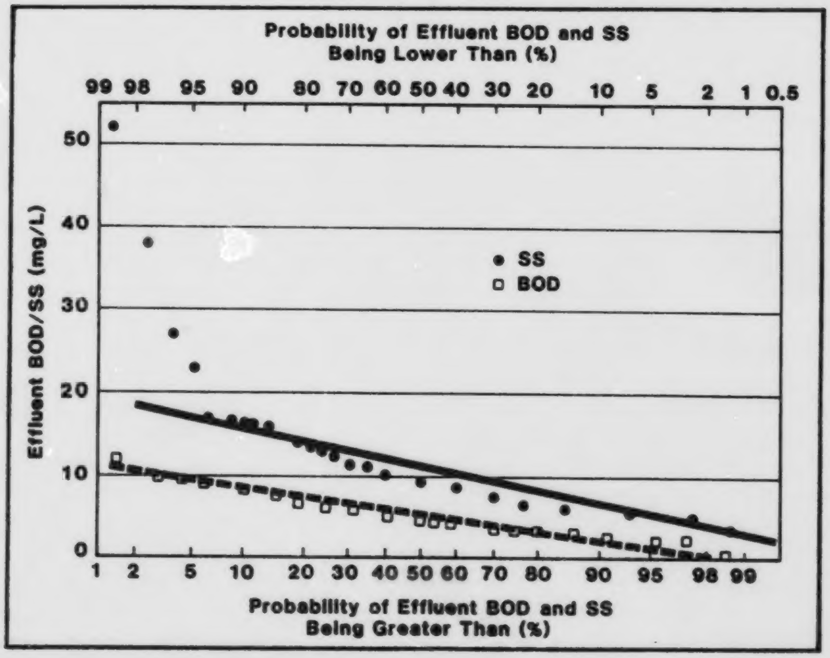
Daily operating data for the 19 month period from June 1984 to December 1985 were reviewed and analyzed to evaluate plant performance. The weekly averages of the plant's operating parameters, such as flow, temperature, influent BOD and suspended solids, organic loading, mixed liquor suspended solids (MLSS) concentration, clarifiers overflow rates and solids loading, sludge volume index (SVI), sludge return rates, and the effluent BOD and suspended solids, were plotted to identify unusual values or occurrences. The continuous data plots for various plant parameters correlated well, indicating a high level of accuracy and reliability of plant records. The continuous data plots were also used for identifying the periods of abnormal or upset operating conditions which lead to further study of the daily logs to establish the reason(s) for the occurrence of upset conditions.

Treatment Effectiveness

Effluent BOD and SS are the two parameters commonly used to assess the performance of the secondary wastewater treatment plants. Figure 3 shows the probability plot of the weekly mean effluent BOD_5 and SS concentration for the 19-month period for which the data were analyzed. The median effluent BOD_5 and SS concentrations for the 19-month period were 5 and 11 mg/L, respectively. The plant effluent quality consistently exceeded the Environmental Protection Service (EPS) guidelines for effluent discharge to surface waters; the guidelines require that 80 percent of the effluent samples taken during a 30-day period must be equal to or better than 20 mg/L BOD and 25 mg/L SS. A review of the plant's daily logs indicated that the effluent BOD was always well below the EPS guidelines; only for one short period during August-September 1984, the effluent SS exceeded the EPS guidelines. As discussed later, the increase in effluent SS was due to an upset condition resulting from low MLSS concentration caused by excessive wasting of the sludge. Otherwise, the plant has successfully endured the sharp seasonal fluctuations in the waste loads and the operating procedure.

FIGURE 3

Probability Plot of the Effluent Quality



Operational Relationships

Table 2 summarizes statistics for plant operational parameters based on operating data for the 19-month period from June 1984 to December 1985. The values of the parameters shown in the table fall within the generally accepted range for the extended aeration activated sludge process.

TABLE 2

Statistics For Operational Parameters from June 1984 to December 1985

Parameter	Mean	Standard Deviation
Hydraulic Retention Time (HRT) - hours	25	5
Organic Loading - kg BOD_5/kg MLSS - day	0.04	0.01
MLSS Concentration - mg/L	2280	460
MLVSS Concentration - mg/L	1760	350
Clarifiers Overflow Rate - $m^3/day/m^2$	11.2	1.9
Sludge Return Rate - % of plant inflow	124	17
Sludge Volume Index	154	70
Sludge Wasting Rate - kg/d	80	80

The parameters, such as hydraulic retention time (HRT) and clarifier overflow rates, are the functions of plant flow in relation to its size; the plant operator has limited or no control over their values. The values of other parameters such as sludge return and sludge wasting rate, however, are controlled by the operator. These parameters have a direct influence on the MLSS concentration level in the aeration basins, which is a critical parameter for the performance of the activated sludge process.

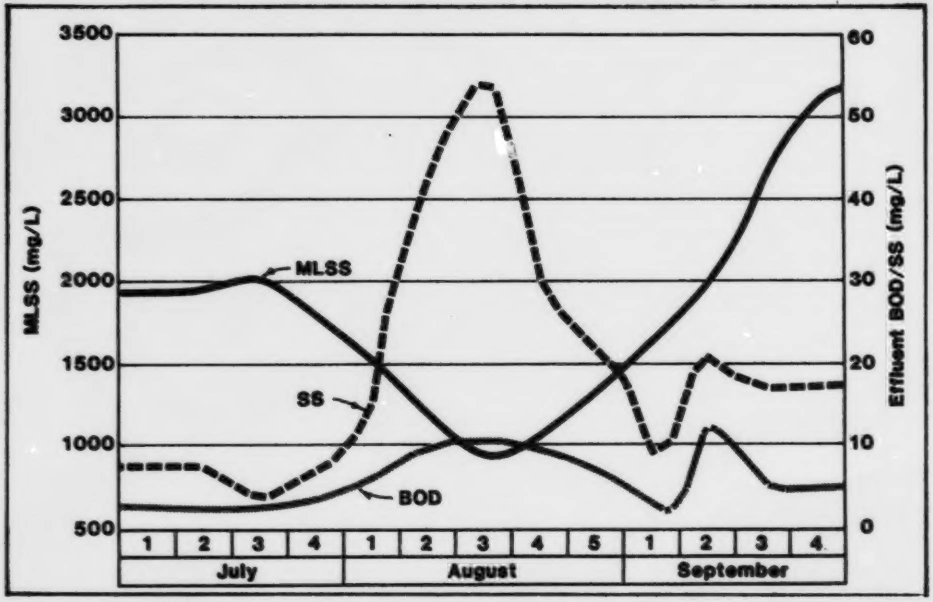
A recent study (2) into the causes of activated sludge treatment plants' process upsets identifies low MLSS concentration as one of the two most frequent causes for poor quality effluent. The study identified the low MLSS upset condition when the organic loading (F:M ratio) as measured by kg BOD_5/kg MLSS-day was within the normally accepted range. Insufficient biomass was also identified as a serious operational problem in an earlier investigation into the performance of extended aeration activated sludge package plants (3). A key parameter for the successful operation of an activated sludge plant, therefore, appears to be the ability to determine and maintain an optimum level of MLSS concentration that will provide assimilation of BOD in the aeration basin and good solids separation in the secondary clarifiers.

For the Lake Louise Wastewater Treatment Plant, the MLSS concentration of 2000 mg/L and higher has consistently provided excellent effluent quality in terms of its BOD and SS concentration. This level of MLSS concentration is consistent with the recommendation contained in an early study (3), that for the successful operation of an extended aeration plant, the average MLSS concentration should not drop below 2000 mg/L.

As mentioned earlier, a plant upset condition due to low MLSS concentration did occur at Lake Louise during August-September 1984. A continuous plot of average weekly MLSS concentration and effluent quality for the period July through September 1984, as shown in Figure 4, clearly indicates a direct relationship between MLSS concentration and the effluent quality. A review of the daily plant logs indicated that the drop in MLSS concentration was triggered by excessive wasting of sludge over a 3-week period starting about the third week in July. The problem of deteriorating effluent quality was detected at the beginning of about the second week in August and the wasting of sludge was stopped in an attempt to increase the MLSS concentration and thus improve the effluent quality. As indicated in Figure 4, the effluent quality returned to its normal level once the MLSS concentration was above 2000 mg/L.

FIGURE 4

Effect of MLSS on Effluent Quality



The organic loading as measured by kg BOD_5/kg MLSS-day during the upset conditions did not exceed 0.07, which is within the normal operating range for the extended aeration activated sludge process. The deterioration in the effluent quality was primarily the result of poor performance of the secondary clarifiers which resulted in excessive SS in the clarifier effluent. The increase in BOD appeared to be primarily related to the SS in the effluent rather than the lack of BOD_5 assimilation in the aeration basins.

COMPARISON OF THEORETICAL DESIGN WITH FIELD DATA

The success of an activated sludge treatment plant design greatly depends upon the ability to predict the organic food (BOD) removal in the aeration basin. The resulting accumulation of sludge in the system due to synthesis of new micro-organisms has to be removed from the process as waste sludge in order to maintain the proper balance between the mass of micro-organisms (MLSS) in the system and the amount of food in the influent wastewater. Several predictive equations are now available to determine the BOD removal rate and the resulting accumulation of excess sludge that needs to be removed from the system. The equations developed by McKinney for designing a complete mixed activated sludge system were used in the design of Lake Louise Wastewater Treatment Plant. This section compares the effluent soluble BOD and sludge production rates determined from the field data with the calculations based on theoretical equations.

BOD Removal

The soluble effluent BOD can be estimated by the following equation provided the aeration basin is not oxygen deficient.

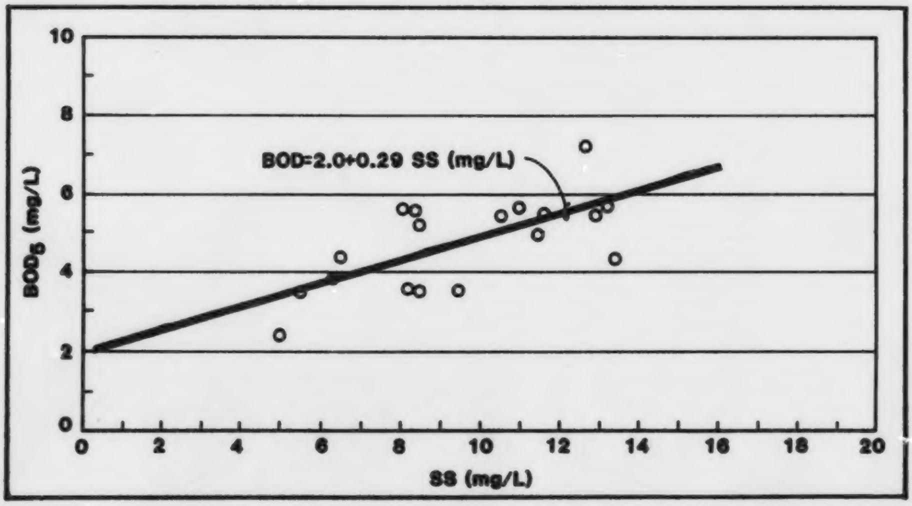
$$F = \frac{Fi}{Km\ t + 1}$$

where F = soluble effluent BOD_5 - mg/L
Fi = influent BOD_5 - mg/L
Km = metabolism factor - day^{-1}
t = aeration time - days.

For metabolism factors of 256/day and 130/days at 15°C and 5°C, respectively, the soluble effluent BOD_5 for the Lake Louise plant was calculated as 0.4 mg/L by using the above equation. While soluble effluent BOD_5 was not measured at the Lake Louise plant, an estimate of the soluble effluent BOD_5 was made by making a linear regression correlation between the effluent BOD_5 and the effluent SS and by taking the effluent BOD_5 at zero SS as the soluble BOD. The correlation for the Lake Louise data is shown in Figure 5. The data for the low MLSS upset condition which occurred during August 1984 was not included in this analysis. The data correlation (r^2) of 0.66 indicates an acceptable degree of correlation considering the diversity of operating variables such as temperature and waste loads. The data indicate a soluble BOD_5 of 2 mg/L. The difference between the two values is not critical considering the precision of the numbers involved. It can, therefore, be concluded that the theoretical equation provides a basis to predict soluble effluent BOD_5 for an activated sludge process.

FIGURE 5

Effluent BOD and SS Relationship



The total effluent BOD is comprised of the soluble effluent BOD and the suspended effluent BOD due to the active microbial mass in the effluent suspended solids. Since the suspended effluent BOD contributes significantly to the total effluent BOD, the efficiency of the secondary clarifiers for removing suspended solids is vital for achieving good quality effluent and should be considered in predicting the effluent BOD.

Sludge Production

The ability to predict the amount of suspended solids that needs to be wasted from an activated sludge process is a critical design consideration. For extended aeration type systems, there is a scarcity of sludge production data due to lack of published information on field installations. Furthermore, any data that are available are based on the treatment of relatively warm sewage (10°C or higher) and are of limited used for cold (less than 5°C) wastewater treatment applications. The sludge production is sensitive to changes in temperature, particularly at lower organic loading rates. Theoretical sludge production estimates for MLSS temperatures of 5°C have been reported to be as much as two to three times the production rates at 20°C (4). Figure 6 is a plot of sludge production in terms of kg SS/kg BOD_5 removed vs. solids retention time (SRT) for the range of wastewater temperatures for Lake Louise. The calculations were based on McKinney's equations for a complete mixed activated sludge system using the following kinetic factors.

	15°C	5°C
Metabolism factor ($km\text{-}day^{-1}$)	256	130
Synthesis factor ($ks\text{-}day^{-1}$)	228	110
Endogenous decay factor ($ke\text{-}day^{-1}$)	0.34	0.17
Influent volatile SS fraction (%)	70	70

FIGURE 6

Effect of SRT and Temperature on Sludge Production

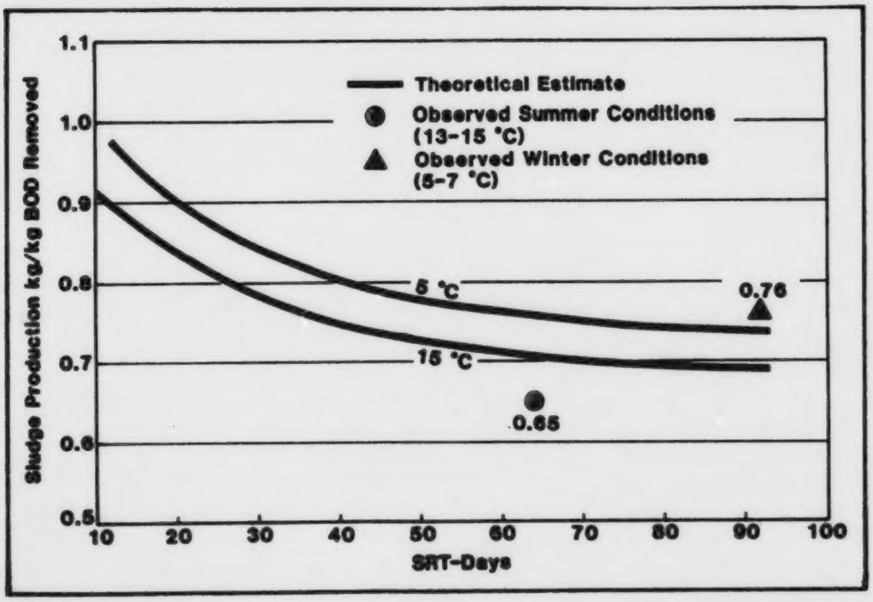


Figure 6 also shows the recorded sludge production rates during summer and winter operations at Lake Louise. These values were based on solids mass balance calculations for periods of continuous operation during which influent wastewater flow, MLSS concentration, and temperature were reasonably uniform. The difference between the predicted and the recorded values is not critical considering the sensitivity of the sludge production data to plant operating conditions and the precision of the analysis techniques involved.

Assuming 30 percent of influent SS to be inert, about 0.4 kg SS/kg BOD_5 removed can be attributed to the inert material in the influent wastewater SS. This would mean, that depending upon the operating temperature, about 0.25 to 0.35 kg VSS/kg BOD_5 removed was produced. These results are within the range of sludge production data for extended aeration systems reported by others [1,5].

CONCLUSIONS

Since it began operation, the Lake Louise Wastewater Treatment Plant has consistently produced high quality effluent exceeding the EPS guidelines relating to effluent BOD_5 and SS concentration. The plant design has successfully endured the sharp seasonal fluctuations in waste loads and operating temperatures. The performance of Lake Louise Wastewater Treatment Plant is indicative of the fact that the activated sludge treatment process can successfully adapt to waste load and temperature fluctuations, provided these problems are adequately identified and addressed during the planning and design stages.

ACKNOWLEDGEMENTS

The author would like to thank the Parks Canada Treatment Plant Operations staff for providing the plant data records.

REFERENCES

1. Environment Protection Service, "Critical Evaluation of Extended Aeration Systems in Arctic and Subarctic Regions". Economic and Technical Review Report EPS 3-WP-77-10. September 1977.

2. Berthouex, P.M. and Fan, R., "Evaluation of Treatment Plant Performance: Causes, Frequency, and Duration of Upsets". Journal WPCF, Volume 58, Number 5. May 1986.

3. Guo, P.H.M., Thirumurthi, D., and Jank, B.E., "An Evaluation of Extended Aeration Activated Sludge Package Plants". Paper presented at the AQTE/FACE (Federation of Associations on the Canadian Environment) Annual Conference, Quebec City. May 1978.

4. Boyl, J.D., "Biological Treatment Process in Cold Climates". Water and Sewage Works. 1976.

5. Pfeffer, J.T., "Extended Aeration Process". Water and Sewage Works. June 1966.

ODOUR CONTROL AT THE CITY OF CALGARY FISH CREEK WASTEWATER TREATMENT PLANT

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Odours from domestic, commercial and industrial establishments have been a problem and a nuisance to man for a long time. A more aware public and the encroachment of subdivisions and commercial businesses on odour sources has brought about the necessity for the control of odorous compounds. The presence of odourants in air which creates an odour response is a frequent source of complaint by the public to operating and regulatory agencies.

Odour is a sensation resulting from stimulation of the olfactory organs and more simply the experience of detecting a smell. The property of a substance that stimulates the sense of smell is called an odourant or osmogene. In order for odour transmission to be complete, a sender, carrier and receiver are necessary. The source is the sender, air currents the carrier and in the case of human beings the nose is the receiver.

ODOUR COMPONENTS IN SEWAGE

Wastewater collection, treatment and disposal systems by nature of the materials handled can be notorious generators of foul odours. Although fresh raw domestic sewage in the diluted condition is not markedly offensive in odour character or intensity, sewer systems and treatment facilities become a source of objectionable odours when they cease to operate under optimum conditions. The majority of the odorous compounds generated characteristically have very disagreeable odours. Table 1 lists some of the odorous air pollutants, together with their odour descriptors and their threshold values. Odour threshold expressed in ppm by volume is exceptionally low for some of the substances encountered in sewerage systems. In addition, it should be noted that for certain substances, a range is given for the threshold concentration. The determination of the threshold number in itself is not an exact science since the human nose is used to determine this property.

Odorous compounds associated with sewage collection and treatment systems may be classified in broad terms as either organic or inorganic. Hydrogen sulfide is one of the major inorganic odours produced in collection and treatment systems. It is formed by the reduction of sulfates in sewage by anaerobic bacteria. Sulfates occur naturally in water and are also formed from the decomposition of urine and protein. Ammonia, the other major inorganic odour, originates primarily from the conversion of urea.

Organic compounds which are prominent foul odour producers include reduced sulfur compounds (mercaptans, thioethers), ammonia derivatives (amines), metabolic products (organic acids, aldehydes, ketones) and the nitrogen compounds indole and skatole.

TABLE 1

Major Odorous Air Pollutants, Olfactory Thresholds and Related Data[1]

Category and Class	Systemic Chemical Name	Formula	Odour	Mol. Wt.	Odour Threshold ppm (by vol.)[1]
Sulfur Compounds:					
Sulfur Oxides	Sulfur Dioxide	SO_2	Pungent	64	0.47
Sulfides	Hydrogen Sulfide	H_2S	Rotten eggs	34	0.0047-0.18
	Carbon Disulfide	CS_2	Rotten	76	0.21-0.84
Mercaptans	Methyl mercaptan	CH_3SH	Decayed Cabbage	48	2×10^{-5}-0.041
	Ethyl mercaptan	C_2H_5SH	Decayed Cabbage	62	3×10^{-5}-0.001
	Propyl mercaptan	C_3H_7SH	Unpleasant	76	0.0016-0.024
	Allyl mercaptan	$CH_2{=}CHCH_2SH$	Garlic	74	0.003-0.017
	Benzyl mercaptan	$C_6H_5CH_2SH$	Unpleasant	124	0.0026-0.04
Thioethers	Dimethyl sulfide	$(CH_3)_2S$	Decayed Cabbage	62	0.003
	Diethyl sulfide	$(C_2H_5)_2S$	Foul, Garlic	90	0.0048
	Diallyl disulfide	$(CH_2{=}CHCH_2S)_2$	Garlic	146	1.1×10^{-4}-0.012
Nitrogen Compounds:					
Inorganic	Ammonia	NH_3	Pungent	17	0.47-54
Aliphatic amines	Dimethylamine	$(CH_3)_2NH$	Fishy	45	0.047
	Trimethylamine	$(CH_3)_3N$	Fishy-ammoniacal	59	0.00021
Aromatic nitro Compounds	2,4,6-Trinitro-t-butylxylene (musk)	$C(C_4H_9)(CH_3)_2(NO_2)_3$	Musk	297	6×10^{-6}-0.005
Heterocyclic amines	Pyridine	C_5H_5N	Empyreumatic	79	0.003-0.23
	Benzo(b)pyrrole (Indole)	C_8H_7N	Fecal	117	
	3-Methylindole (skatole)	C_9H_9N	Fecal	131	0.05
Cyanides	Hydrogen cyanide	HCN	Bitter Almonds	27	0.9
	Allylisocyanide	$CH_2{=}CHCN_2NC$	Sweet Repulsive (nauseating)	67	0.18-1.6
	Allylisothiocyanate	$CH_2{=}CHCH_2SNC$	Mustard Oil (nose & eye irritant)	99	0.008-0.42
Selenium Compounds:					
Selenides	Hydrogen Selenide	H_2Se	Putrid	81	4×10^{-4}-0.0012
	Ethylselenomercaptan	C_5H_5SeH	Foul, Fetid	109	4×10^{-4}-0.0012
	Diethyl selenide	$(C_2H_5)_2Se$	Putrid	137	0.011

Category and Class	Systemic Chemical Name	Formula	Odour	Mol. Wt.	Odour Threshold ppm (by vol.)[1]
Hydrocarbons, Alcohols and Oxygenates:					
Aliphatic Hydrocarbons	2-Butene (butylene)	$CH_3CH{=}CHCH_3$	Gas-house	56	24
	2-Methylpropene (isobutylene)	$CH_2{=}C(CH_3)_2$	Gas-house	56	20
Phenol	Phenol	C_6H_5OH	Empyreumatic	94	0.047
Aldehydes:	Methanal (formaldehyde)	H_2CO	Pungent	30	1.0
	Ethanal (acetaldehyde)	CH_3CHO	Pungent	44	0.066-2.2
	Propenal (acrolein)	$CH_2{=}CHCHO$	Burning Fat	56	0.021-1.8
	4-Hydroxy-3-methoxy-benzaldehyde (vanillin)	$C_8H_8O_3$	Sweet-aromatic	152	1.1×10^{-4}-2×10^{-7}
Ketones:	d-2-Keto-1,7,7-trimethylnorcamphene (camphor)	$C_{10}H_{16}O$	Aromatic-earthy	152	1.3
Organic Acids	Butanoic Acid (butyric acid)	$CH_3CH_2CH_2COOH$	Rancid, perspiration	88	0.001-2.2
	2 Methylbutanoic Acid (isovaleric Acid)	$(CH_3)_2CHCH_2COOH$	Body Odour	102	0.015
	Butanediene (diacetyl)	$(CH_3CO)_2$	Sweet Butter	86	0.025
Halogen Compounds:					
Inorganic	Chlorine	Cl_2	Pungent	71	0.31
Aliphatic Halogens	Trichloroethylene	$CHCl{=}CCl_2$	Aromatic	131	0.21
	Triiodomethane (iodoform)	CHI_3	Antiseptic	394	5×10^{-3}
Aromatic Halogens	Benzylchloride	$C_6H_5CH_2Cl$	Aromatic Lacrimator	126.5	0.04-0.31
	Chlorohydroxybenzene (chlorophenol)	$C_6Cl_4(OH)Cl$	Medicinal	128.5	0.0036-0.03
Miscellaneous Oxygen	Trioxygen (ozone)	O_3	Irritating	48	0.51

[1] Data from Fazzalari. ASTM D48A

ODOUR SOURCE

In light of our subsequent discussion on odour sources at the Fish Creek Plant, it is appropriate at this time to briefly review how the odour control program at Fish Creek evolved.

In 1959, the City of Calgary constructed its second wastewater treatment plant at the confluence of Fish Creek and the Bow River, an area in the river valley well removed from residential development and surrounded by meadows used for cattle grazing by a nearby ranch. A major expansion in 1978 increased the plant capacity to 18 mgd and upgraded the plant to include secondary treatment facilities. By this time the picture had changed. The province was now developing the Fish Creek Provincial Park to the south and east of the site and prime residential subdivisions were being developed approximately 0.25 km to the west and north of the plant perimeter. Plant design took into account the sensitive nature of the plant location. Special emphasis was put on aesthetics, noise control and treatment process selection. However, extensive odour control facilities were not included at this time. This decision was based on the premise that a well designed and operated plant using the most recent treatment technology should not cause an odour problem. This was not to be the case. Soon after the 1980 start-up of the expanded facilities, the first of a series of complaints from area residents were received which led to the initiation of this investigation.

In a sewerage system, sewers, pumping stations, syphons and individual processes within the treatment plant are potential sources of odour. All these component sources should be examined to determine their relative significance in preparing an overall odour control strategy. In the case of the Fish Creek Wastewater Treatment Plant, a survey of wastewater contributors to the sewer system was undertaken and discussions concerning sewer design criteria were held with City of Calgary Sewer Division.

The sewage treated by the Fish Creek Plant is primarily domestic. Two major industries which discharge to the Fish Creek collection system are a rendering plant and a potato chip plant. Both plants pretreat their wastewater discharges at the respective plant sites to remove particulates and grease. Soluble organics constitute the bulk of the waste loading from these sources. Occasionally slugs of strong waste have arrived at the City plant and this has resulted in an increase in odours emitted from the primary portion of the treatment plant. Staff with the City of Calgary Sewer Design Department indicate that all sewer sections have been designed with cleansing velocities.

With the exception of periodic odour complaints, the Fish Creek Plant has operated relatively trouble free, consistently producing a high quality effluent. The plant is a full secondary treatment plant utilizing the Unox oxygen activated sludge process and anaerobic sludge digestion. A description of odour sources is provided in the order that process units occur through the plant.

The headworks was considered to be the worst offender, followed by the primary clarifiers and the anaerobic digestion system. Sewage flows to the headworks by gravity where it is screened and then aerated in chambers to facilitate grit removal. Odorous gases are released during the aeration process at the grit tanks and in the turbulent re-aeration section as the wastewater cascades over the weirs to the effluent channel. In addition, odorous air enters the headworks building from the gravity trunk sewer. Screenings and grit are contained in hoppers and are also a source for odour generation. However, since City operating personnel remove screenings daily, odourant generation from this source is minimized.

The pretreated wastewater from the headworks flows in a covered conduit to a splitting chamber in the centre of the primary clarifiers. At the primaries this flow is split at the parshall flumes and flows into the centre of each clarifier via individual pipes.

The space over the wastewater in the conduit contains some odorous gases from the headworks and some of this can be exhausted to the open air at control gates and at the flume measurement devices. Further release of entrained gases to the atmosphere can occur as the wastewater rises in the centre well at the clarifiers and from the anaerobic decomposition of sludge deposits at the bottom of the clarifier.

The second treatment process was not considered an odour generator since the tanks are totally enclosed and utilize pure oxygen instead of air. Exhaust gases from this process are essentially excess oxygen, carbon dioxide and water vapour. The secondary clarification process was considered not to contribute to the odour problem.

The remainder of the processes associated with sludge handling, digestion and utilization of waste digester gas are an area of concern relative to odour generation. These processes are sludge thickening, digestion, boiler plant and waste gas flares.

The sludge thickening process, comprised of centrifuges, was commissioned subsequent to the site monitoring program conducted for odour characterization. A dry scrubber was installed at the sludge thickening building as part of the construction contract to remove the odorous gases.

Odorous emissions from the digestion process are periodically released from the pressure relief valves located on the digester dome, the overflow boxes utilized for transferring digested sludge, the digester gas compressor room to a lesser extent, the exhaust from the boilers and the flaring of excess gas at the four flares provided for the plant.

Produced digester gas is transferred from the sludge digestion complex to a meter room in the boiler plant. Under normal operations, most digester gas is utilized by the boiler system as primary heating fuel, while excess gas volumes are relieved via two pressure regulating valves to the waste gas flares.

Difficulties experienced with the operation of the boiler control system, digester gas boosters for boiler feed and the pressure regulation valves for regulation of digester gas between the boilers and flare stacks has resulted in the frequent exhausting of digester gas to the atmosphere via the digester roof mounted gas relief valves. A further problem is the relatively high pressure settings required on the pressure regulating valves to ensure sufficient digester gas available to the boilers.

SITE MONITORING PROGRAM

Once the odour sources were identified, a monitoring program was designed and implemented to characterize the odour emissions.

The site monitoring program was conducted in two phases. Phase I monitoring was undertaken March 13 and 14, 1985. This involved sensory and analytical measurements on site at specific locations and a community odour survey. Phase II monitoring was conducted from July 10 to August 2, 1985, during warm ambient summer temperatures. Phase II monitoring involved sensory and instrumental monitoring at specific locations at the treatment plant site, including the performance evaluation of an odour control pilot unit located at the headworks building.

The sensory property most commonly measured is detectability via a via dilution of the odorous air with odour free air, until either the detection or recognition thresholds are reached. The detection threshold is that point where a panelist familiar with the odour character can detect the odour 50% of the time. The recognition threshold is that point where a panelist familiar with the odour character can recognize the odour 50% of the time. The odourant concentration at the recognition threshold generally exceeds the detection threshold by a factor of 1.5 to 10.

There are many techniques used to measure detectability or dilution to threshold ratio. All of them involve diluting the odorous sample with odour free air in known ratios. The sensory techniques used in the Fish Creek odour evaluation study were the scentometer and the modified syringe dilution method. The odour intensity of the collected samples were compared to the n-butanol odour intensity scale.

Chromatographs were set up on site to allow for quick and accurate characterization of the samples collected. Reduced sulfur components were of primary interest, but the analysis also included a complete identification of common fixed gases and hydrocarbons of up to five carbon molecules.

Produced gas, relief valve building gases, digester off-gas and pilot plant samples were sampled by hand aspirating the gas through 3 ft of 1/4 inch teflon tubing and bubbling it through P_2O_5 power driers into a 125 mL glass sample envelope. The headworks building, truck dock exhaust and cutwater gas samples were taken in a similar manner. A vacuum pump was connected to the exit of the glass sample system to ensure that the 30 ft. of 1/4 inch teflon tubing and the sample envelope were well purged.

For the boiler stack gas and all the flare samples, gas was drawn by hand aspirator down a 9 ft., 1/2 inch glass tube, followed by 12 ft of 1/4 inch teflon through the driers in the sample envelope.

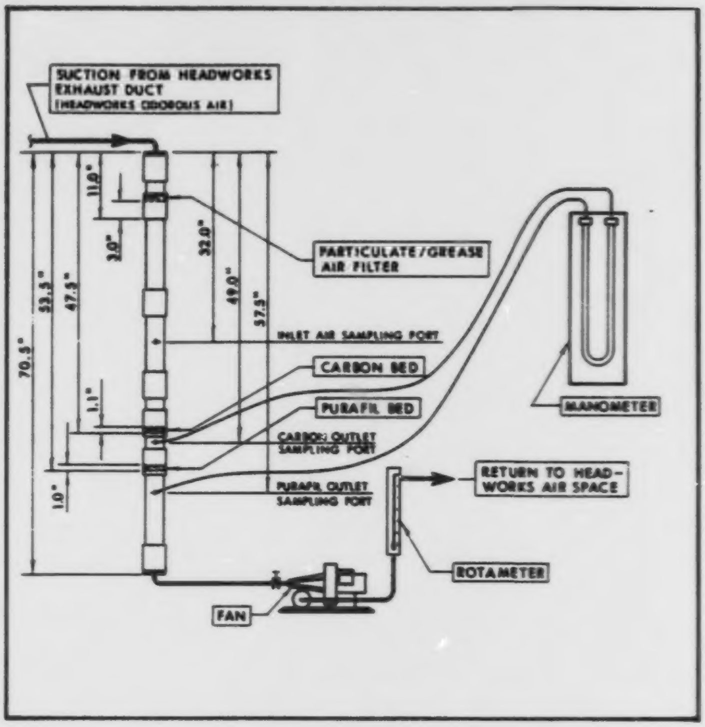
In addition to monitoring the odours emanating from unit treatment processes and from specific locations on the digester gas system, the performance of a pilot adsorption column was monitored as well. The pilot unit was set up to better predict the adsorption behavior of activated carbon and activated alumina and to predict lifetime of the adsorption media in a full scale unit. In the case of municipal and industrial wastewater treatment plants, odour is caused by a complex mixture of organic compounds whose adsorption behaviour is difficult to predict. The adsorption capacity of carbon and its useful lifetime are limited by a number of factors, primarily the nature of contaminants and their concentration and the temperature and humidity of the air stream.

The pilot adsorption unit in Phase II monitoring program for the activated carbon-alumina pilot tests consisted of PVC pipe with an inner diameter of 3 1/16 inch. A carbon bed depth of 1.1 inch, followed by a 1.0 inch bed depth of activated alumina (Purafil) was used. The air was drawn through the pilot unit by a small fan and the flow rate was measured by means of a rotameter. The tests were conducted at an air flow rate of 4 ACFM through the adsorption train. Figure 1 schematically depicts the pilot plant components.

The results of the analytical analysis are shown on Tables 2 and 3 for Phase I and Tables 4, 5 and 6 for Phase II monitoring programs respectively. The instrumental analysis provided concentrations of hydrogen sulfide (H_2S) and six low molecular weight organic reduced sulfur compounds. Total reduced sulfur (TRS), representing the sum of all the individual concentrations and the total concentration of the organic reduced sulfur compounds, are presented in separate columns. These two totals were provided to facilitate correlation between the concentration of the odourants and the odour concentration determined by sensory techniques.

FIGURE 1

City of Calgary
Fish Creek Sewage Treatment Plant
Schematic of the Activated Carbon - Purafil
Pilot Test Equipment


SUCTION FROM HEADWORKS EXHAUST DUCT (HEADWORKS ODOROUS AIR)
11.0"
2.0"
32.0"
53.5"
47.5"
49.0"
57.5"
70.5"
PARTICULATE/GREASE AIR FILTER
INLET AIR SAMPLING PORT
CARBON BED
PURAFIL BED
1.1"
CARBON OUTLET SAMPLING PORT
1.0"
PURAFIL OUTLET SAMPLING PORT
MANOMETER
RETURN TO HEAD-WORKS AIR SPACE
ROTAMETER
FAN

During the winter testing period the total reduced sulfur concentrations ranged from 103 to 2,129 ppb for most sources. The only exception was the produced gas generated in sludge digestion, for which the TRS values ranged from 2,861 ppb to 496,000 ppb, in large part due to high concentration of H_2S. The odour concentrations for most sources ranged from 24 to 1,000 dilutions to threshold (D/T). As expected, the concentration for the produced gas is higher, 5,000 D/T.

Neither the total nor the organic TRS values appear to show correlation with the odour concentrations. The lack of correlation significantly surpasses the errors for both analytical and sensory methods. The lack can be explained by the presence of some odourants, possibly very low odour threshold, which were not detected by the instrumental techniques.

The TRS values obtained during the summer tests are in the range of 8 to 5,360 ppb, excluding the produced gas values and the results of the pilot test unit on August 2. Again, no correlation is apparent between the sensory and analytical data. The emphasis of the sensory evaluation during the summer was on pilot tests.

TABLE 2

Analytical and Sensory
Odour Analysis Summary For Selected Treatment Processes
Phase I - March 13, 1985

			Pollutant Concentration (ppb)										
			Mercaptans (1)										Odour Concentration
Date	Time	Sample Location	H_2S	C_1	C_2	C_3	C_4	DMS (2)	DMDS (3)	TRS (4)	TRS-H_2S	SO_2	D/T (5)
13/03/85	11:57	Headworks - Building East Fan Outlet	1352	32	ND	ND	165	319	261	2129	777	ND	-
13/03/85	13:04	Headworks - Building West Fan Outlet	913	T	NA	NA	NA	NA	NA	-	-	ND	-
13/03/85	13:44	Grit Tank - 6" Above Water at Center	185	9	ND	ND	183	240	262	-	-	ND	-
13/03/85	14:11	Grit Tank - 6" Above Water at Center	644	7	NA	NA	NA	NA	NA	-	-	ND	-
13/03/85	14:37	Headworks - West Sewage Inlet Upstream of Coarse Screen	594	5	ND	ND	160	267	215	1241	647	ND	25
13/03/85	15:04	Headworks - East Sewage Inlet Upstream Coarse Screen	496	14	ND	ND	252	53	97	912	416	ND	50
13/03/85	15:24	Upstream Headworks Above Cut Water Gate	511	18	ND	ND	134	132	162	957	446	ND	-
13/03/85	16:17	Produced Gas	492,000	588	67	299	2796	81	ND	495,831	3,831	ND	5,000
13/03/85		Liquid Sample Headworks Inlet	ND	< 1	3	T	43	9	47	103	103	ND	-

NOTES:

T - Trace, less than 0.1 ppb
ND - Not detected
NA - Not Analysed

(1) C_1 = methyl mercaptan (CH_3S); C_2 = ethyl mercaptan (C_2H_5S); C_3 = propyl mercaptan (C_3H_2S); C_4 = butyl mercaptan (C_4H_9S)
(2) DMS = dimethyl sulfide
(3) DMDS = dimethyl disulfide
(4) TRS = total reduced sulfur
(5) D/T = dilution to threshold

TABLE 3

Analytical and Sensory
Odour Analysis Summary For Selected Treatment Processes
Phase I - March 14, 1985

			Pollutant Concentration (ppb)										Odour Concentration
				Mercaptans(1)									
Date	Time	Sample Location	H_2S	C_1	C_2	C_3	C_4	DMS(2)	DMDS(3)	TRS(4)	TRS-H_2S	SO_2	D/T(5)
14/03/85	8:05	Headworks Building - East Fan Outlet	345	< 1	1	T	114	96	47	604	259	ND	37
14/03/85	9:15	Headworks Building - West Fan Outlet	548	2	ND	T	119	24	43	736	188	ND	-
14/03/85	9:30	Liquid Sample - Headworks Inlet	607	39	8	4	138	9	38	843	236	ND	1,000
14/03/85	9:30	Liquid Sample - Grit Tank Outlet	ND	11	4	3	123	5	78	224	224	ND	1,000
14/03/85	10:52	Produced Gas	251	37	174	121	408	82	57	1,130	879	ND	-
14/03/85	11:36	Upstream Headworks Above Cut Water Gate	1,067	1	ND	ND	ND	245	493	1,806	739	ND	375
14/03/85	11:43	Headworks - East Sewage Inlet Upstream Coarse Screen	1,053	< 1	ND	ND	74	270	76	1,474	421	ND	750
14/03/85	12:32	North Boiler Stack Gas	ND	< 1	ND	ND	153	15	9	178	178	41	-
14/03/85	12:56	Holding Tank - #3 Digester	ND	160	124	29	249	29	39	630	630	ND	-
14/03/85	13:03	#4 Digester Relief Valve Building	72,000	13	9	ND	74	158	230	72,484	484	ND	-
14/03/85	13:47	South East Flare	422	12	ND	ND	55	74	12	575	153	9,000	-
14/03/85	14:15	#2 Digester	237	66	141	63	267	49	38	861	624	ND	-
14/03/85	14:45	#2 Digester Relief Valve Building	174,000	1	24	T	157	45	291	174,518	518	ND	-

NOTES:

T - Trace, less than 0.1 ppb
ND - Not detected
NA - Not Analysed

(1) C_1 = methyl mercaptan (CH_3S); C_2 = ethyl mercaptan (C_2H_5S); C_3 = propyl mercaptan (C_3H_2S); C_4 = butyl mercaptan (C_4H_9S)
(2) DMS = dimethyl sulfide
(3) DMDS = dimethyl disulfide
(4) TRS = total reduced sulfur
(5) D/T = dilution to threshold

TABLE 4

Analytical and Sensory
Odour Analysis Summary For Selected Treatment Processes
Phase II - July 9 and 10, 1985

			Pollutant Concentration (ppb)										
				Mercaptans(1)									
Date	Time	Sample Location	H_2S	C_1	C_2	C_3	C_4	DMS(2)	DMDS(3)	TRS(4)	TRS-H_2S	SO_2	Odour Concentration D/T(5)
09/07/85	8:15	S.E. Flare	69	1	2	ND	ND	15	ND	87	18	ND	88
09/07/85	9:53	South Boiler Stack	335	10	ND	ND	ND	ND	ND	345	10	ND	-
09/07/85	10:12	Produced Gas	690	21	ND	ND	ND	ND	ND	711	21	ND	-
09/07/85	11:27	S.E. Flare	525	191	ND	ND	ND	ND	ND	716	191	9	-
09/07/85	11:59	S.W. Flare	5,135	245	ND	ND	ND	ND	ND	5,380	245	24	20
09/07/85	12:55	South Boiler Stack	ND	ND	ND	ND	ND	ND	ND	ND	ND	9	-
09/07/85	13:07	Produced Gas	244	106	ND	ND	ND	11	ND	361	117	ND	-
10/07/85	11:00	Headworks Building - West Fan Outlet	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	-
10/07/85	11:52	#3 Holding Tank	ND	163	18	ND	ND	ND	ND	181	181	ND	-
10/07/85	12:03	#4 Digester Relief Valve Building	67	15	37	ND	ND	ND	ND	119	52	ND	-
10/07/85	12:59	Headworks West Inlet Upstream Coarse Screen	T	ND	127	ND	ND	ND	279	406	406	ND	-
10/07/85	13:04	Upstream Headworks Above Cut Water Gate	2,039	7	235	ND	ND	ND	ND	2,281	242	ND	-
10/07/85	14:07	#4 Digester	66,000	343	38	ND	ND	ND	ND	66,381	381	ND	-
10/07/85	15:05	#2 Digester Relief Valve Building	2,012	31	25	ND	ND	ND	ND	2,068	56	ND	-
10/07/85	15:27	Upstream Headworks Above Cut Water Gate	643	25	126	ND	ND	ND	ND	794	151	ND	-

NOTES:

T - Trace, less than 0.1 ppb
ND - Not detected
NA - Not Analysed

(1) C_1 = methyl mercaptan (CH_3S); C_2 = ethyl mercaptan (C_2H_5S); C_3 = propyl mercaptan (C_3H_2S); C_4 = butyl mercaptan (C_4H_9S)
(2) DMS = dimethyl sulfide
(3) DMDS = dimethyl disulfide
(4) TRS = total reduced sulfur
(5) D/T = dilution to threshold

TABLE 5

Analytical and Sensory
Odour Analysis Summary For Selected Treatment Processes
Phase II - July 11 and 12, 1985

			Pollutant Concentration (ppb)										Odour Concentration
				Mercaptans[(1)]									
Date	Time	Sample Location	H_2S	C_1	C_2	C_3	C_4	DMS[(2)]	DMDS[(3)]	TRS[(4)]	TRS-H_2S	SO_2	D/T[(5)]
11/07/85	8:25	S.E. Flare	487	16	1	ND	ND	1	ND	505	18	ND	-
11/07/85	8:53	S.W. Flare	ND	1	26	ND	ND	1	ND	28	28	ND	-
11/07/85	9:57	Truck Dock Exhaust	14	T	11	ND	ND	ND	ND	25	11	ND	-
11/07/85	11:25	S.W. Flare	ND	ND	8	ND	ND	ND	ND	8	8	ND	-
11/07/85	11:32	S.E. Flare	32	4	T	ND	ND	8	ND	44	12	ND	-
11/07/85	14:43	South Boiler Stack - Low Fire	31	1	28	ND	ND	ND	ND	60	29	ND	-
11/07/85	15:25	South Boiler Stack - High Fire	T	3	45	ND	ND	ND	ND	48	48	ND	-
11/07/85	16:13	S.E. Flare - High Fire	ND	39	19	16	ND	54	ND	128	128	25	-
11/07/85	11:22	S.E. Flare - Low Rate	ND	3	20	ND	ND	17	ND	40	40	ND	-
11/07/85	16:48	Produced Gas	4,070	446	16	ND	ND	T	ND	4,532	462	ND	-
12/07/85	8:56	Produced Gas	36,000	166	5	ND	ND	ND	ND	36,171	171	ND	-
12/07/85	11:17	South Boiler Stack - Low Fire	21	1	7	ND	ND	ND	ND	29	8	25,000	-
12/07/85	11:21	South Boiler Stack - High Fire	26	1	28	ND	ND	ND	ND	55	29	14,000	-

NOTES:

T - Trace, less than 0.1 ppb
ND - Not detected
NA - Not Analysed

(1) C_1 = methyl mercaptan (CH_3S); C_2 = ethyl mercaptan (C_2H_5S); C_3 = propyl mercaptan (C_3H_2S); C_4 = butyl mercaptan (C_4H_9S)
(2) DMS = dimethyl sulfide
(3) DMDS = dimethyl disulfide
(4) TRS = total reduced sulfur
(5) D/T = dilution to threshold

TABLE 6

Analytical and Sensory
Odour Analysis Summary For Pilot Plant Operation
Phase II - July 10 to August 2, 1985

Date	Time	Sample Location	Pollutant Concentration (ppb)									Odour Concentration D/T$^{(5)}$
			Mercaptans$^{(1)}$									
			H_2S	C_1	C_2	C_3	C_4	DMS$^{(2)}$	DMDS$^{(3)}$	TRS$^{(4)}$	TRS-H_2S	
10/07/85	16:40	Pilot Unit Inlet	153	2	54	ND	ND	ND	ND	209	56	70
10/07/85	16:40	Carbon Bed Outlet	ND	14	46	ND	ND	ND	ND	60	60	10
10/07/85	16:40	Purafil Bed Outlet	ND	9	28	ND	ND	ND	ND	9	9	5
11/07/85	8:55	Pilot Unit Inlet	342	5	4	ND	ND	ND	ND	351	9	150
11/07/85	8:52	Carbon Bed Outlet	ND	ND	T	ND	ND	ND	ND	0	0	40
11/07/85	9:02	Purafil Bed Outlet	ND	ND	T	ND	ND	ND	ND	0	0	15
11/07/85	12:00	Pilot Unit Inlet	69	17	71	ND	ND	ND	ND	157	88	500
11/07/85	12:00	Carbon Bed Outlet	ND	ND	20	ND	ND	ND	ND	20	20	75
11/07/85	12:00	Purafil Bed Outlet	ND	ND	15	ND	ND	ND	ND	15	15	15
12/07/85	9:54	Pilot Plant Inlet	30	8	31	ND	ND	ND	ND	69	39	500
12/07/85	9:58	Carbon Bed Outlet	ND	9	12	ND	ND	ND	ND	21	21	20
12/07/85	10:01	Purafil Bed Outlet	ND	11	10	ND	ND	ND	ND	21	21	10
12/07/85	12:14	Pilot Plant Inlet	21	7	54	ND	ND	ND	ND	82	61	-
12/07/85	12:17	Carbon Bed Outlet	ND	4	12	ND	ND	ND	ND	16	16	-
12/07/85	12:20	Purafil Bed Outlet	ND	ND	16	ND	ND	ND	ND	16	16	-
02/08/85	10:42	Pilot Unit Inlet	ND	ND	1760	2290	2030	3430	(6)	9510	9510	500
02/08/85	10:50	Pilot Plant Inlet	ND	ND	4200	6660	0190	3520	(6)	14570	14570	500
02/08/85	10:40	Carbon Bed Outlet	ND	ND	2220	5860	0190	4690	(6)	12960	12960	500
02/08/85	10:48	Carbon Bed Outlet	ND	ND	8390	14180	1570	14980	(6)	39120	39120	500
02/08/85	10:35	Purafil Bed Outlet	ND	ND	0470	2570	0720	(6)	(6)	3760	3760	500
02/08/85	10:45	Purafil Bed Outlet	ND	ND	3170	8070	0210	4960	(6)	16410	16410	500

NOTES:

T - Trace, less than 0.1 ppb
ND - Not detected
NA - Not Analysed

(1) C_1 = methyl mercaptan (CH_3S); C_2 = ethyl mercaptan (C_2H_5S); C_3 = propyl mercaptan (C_3H_2S); C_4 = butyl mercaptan (C_4H_9S)
(2) DMS = dimethyl sulfide
(3) DMDS = dimethyl disulfide
(4) TRS = total reduced sulfur
(5) D/T = dilution to threshold
(6) Unable to determine due to interference

There is no definitive explanation why the August 2 results are markedly different than previous results. The blow for the pilot test unit seized up a few hours prior to the final sampling. This may have affected the final samples subsequently collected on August 2 after installing a second blower for purposes of providing a sample.

Surveys conducted by Henry and Gehr (University of Toronto) in 1975 of 142 wastewater treatment plants in Ontario identified odourants similar to those documented during the Fish Creek study and in addition, others such as indole, skatole and organic acids. The major sources for these odours in the Ontario survey were attributed to sludge handling facilities, diffused aeration systems, primary settling facilities and anaerobic treatment. A similar survey conducted by Ledbetter in 1984 of U.S. plants identified similar odourants as being most responsible for creating odours. No concentration values were quoted for these odourants identified in these surveys.

The pilot test ran continuously for 22 days until a complete breakthrough of both carbon and activated alumina beds was achieved. Odour concentrations were determined at the inlet and outlets of the carbon and purafil beds. The data are presented in Figure 2 and Table 7. The inlet odour concentration showed some fluctuation, but for a significant portion of the test it varied within a narrow range of 400 to 500 dilutions to threshold. The odour emission concentration from both the carbon and purafil beds was constantly increasing throughout the test.

FIGURE 2

City of Calgary
Fish Creek Sewage Treatment Plant
Activated Carbon - Purafil Pilot Test
Odour Breakthrough

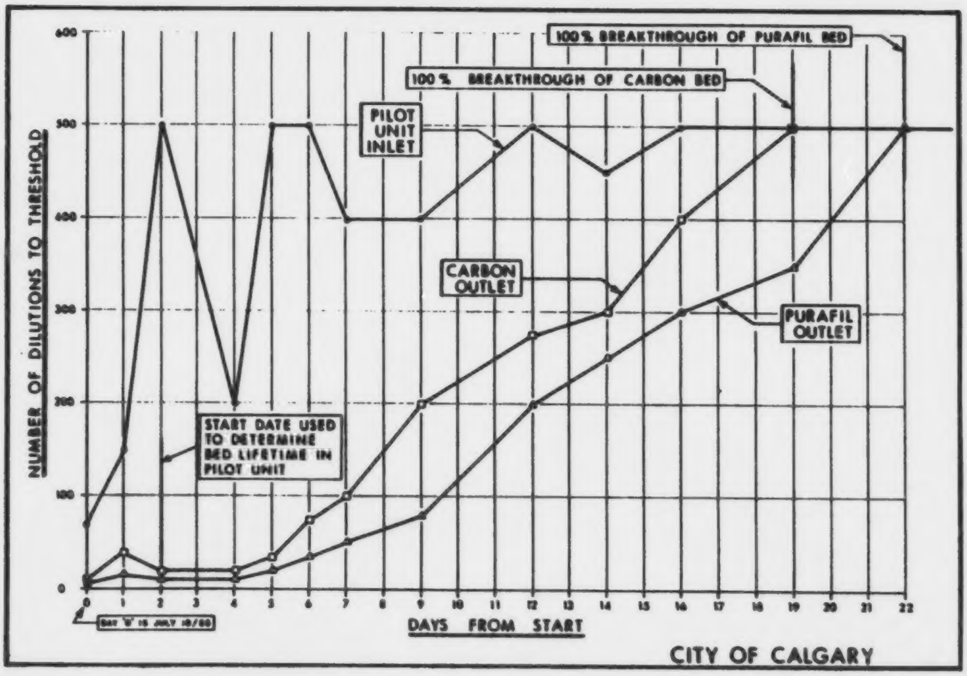


TABLE 7

Calculation of Total Quantity of Odour Controlled

Time		Odour Concentration (D/T)*			Gas Flow Rate (ACFH)	Odour Emmissions$^{(1)}$ (O.U. x 10^6)**			Odour Retained (O.U. x 10^6)**	
Interval	Hours	In	Carbon Outlet	Purafil Outlet		Inlet	Carbon Outlet	Purafil Outlet	On Carbon	On Purafil
1	48	350	20	10	255	4.284	0.245	0.124		
2	24	350	30	15	250	2.100	0.180	0.09		
3	24	500	60	25	243	2.916	0.350	0.146		
4	24	450	90	40	240	2.592	0.518	0.230		
5	48	400	150	65	238	4.570	1.714	0.743		
6	72	450	240	140	235	7.614	4.061	2.369		
7	48	475	290	225	233	5.312	3.243	2.970		
8	48	475	350	275	230	5.244	3.864	3.036		
9	72	500	450	325	230	8.280	7.452	5.382		
10	72	500	500	425	225	8.100	8.100	6.885		
Totals$^{(2)}$						42.912	21.627		21.285	
Totals$^{(3)}$						51.012	29.727			7.752

*D/T = dilution to threshold; **O.U = odour units, a product of gas flow rate and dilution to threshold.

(1) = average odour emissions in the corresponding time interval.

(2) = total for the first 9 intervals (concludes with 100% carbon breakthrough).

(3) = total for all 10 intervals (concludes with Purafil breakthrough).

Several community surveys were conducted during the winter and summer testing periods. The meteorological conditions during both winter and summer observations were in general favourable for odour dispersion. Odour detection threshold at plant boundary was detected at a very low intensity on a few occasions.

ODOUR CONTROL TECHNIQUES

The most common approach utilized in the past for controlling odours in a sewage treatment facility has been through chemical addition. The often used chemicals are oxidizers (ozone, chlorine, hydrogen peroxide), chemicals that raise the oxidation reduction potential (oxygen, chlorine, nitrate, etc.), bactericides (ozone, chlorine, chlorine dioxide, etc.), and pH modifiers such as lime. Chemical addition has not proved to be totally reliable due to the complexity and variability of potential odour producing constituents in sewage.

Other approaches which employ add-on techniques are used to remove odour from air streams based on a variety of principles. Most commonly employed are incineration, adsorption (wet scrubbing) and adsorption (dry scrubbing). The effectiveness of wet and dry scrubbing may be augmented by chemical reactions.

Unless incineration is performed in conjunction with useful heat recovery, the main disadvantage of this technique is its high cost. The cost could either be the operating cost, primarily for the fuel, or the capital cost for heat recovery.

Wet scrubbing can be used for combined control of low molecular weight acidic reduced sulfur compounds (primarily H_2S) and of other organic odourants. Since the efficiency of wet scrubbing for removal of odourants depends largely on their solubility in water, chemical reactions are necessary to enhance their removal. Oxidation with hypochlorite is commonly employed for that purpose. Sodium hydroxide is added in conjunction with hypochlorite to maintain the pH of the scrubbing liquor above 8. Wet scrubbing has enjoyed some popularity as a technique for odour control in sewage treatment plants. Odour control efficiency of a wet scrubber system could deteriorate quickly if the system is not operated and maintained properly. Typical operating problems are associated with control systems related to addition of chemical reagents.

Dry scrubbers, the adsorption of odourants on solid media, has been used extensively in odour control. In wastewater treatment plants, the odours are to a large degree associated with hydrogen sulfide and other low molecular weight reduced sulfur compounds. Activated carbon is the least effective in controlling these odours. In general, activated carbon is more effective for removing higher molecular weight odourants.

Activated carbon used in series with activated alumina impregnated with potassium permanganate is gaining popularity as an odour control process for municipal wastewater treatment plants. Due to the high oxidative power of potassium permanganate, the medium is well suited for control of low molecular weight reduced sulfur compounds. Its effectiveness in controlling the higher molecular weight compounds is lower. These properties make activated alumina impregnated with potassium permanganate a suitable complement to activated carbon. A pilot test performed at the Fish Creek Wastewater Treatment Plant has demonstrated the effectiveness of this media combination for odour control at the Fish Creek plant.

The system is simple to operate and maintain. Odour removal efficiencies in excess of 95% can be obtained in a given period of time, depending on the frequency of media replacement. The efficiency of the system deteriorates with time, requiring periodic replacement of the media. Logistical problems in media replacement must be resolved and the labour must be provided for replacement. High moisture interferes with performance of both activated carbon and purafil. Therefore, the system must be operated at sufficiently high temperature for the relative humidity to be preferably less than 80%. Where low air volumes are to be treated, dilution of odorous air with outside air during low ambient air temperatures is considered by some to be an option to heating. This is currently being tried at Fish Creek on the sludge thickening process.

Another approach to odour control is to modify unit process operation and facility design in order to minimize the generation and subsequent release of odours. Due consideration should be given to such factors as the waterfall effect in channel flow, accumulation of solids in dead zones within tankage, infrequent sludge pumping, utilization of odorous exhaust gases from primary plant for secondary aeration, more uniform sludge pumping to digesters and the proper selection of equipment.

EVALUATION OF HEADWORKS ODOUR CONTROL

The Fish Creek headworks building was identified from the monitoring program as the major odour source, followed by the primary clarifiers. Other potential odour sources identified are of a more intermittent nature. Process and operation modifications, together with add-on techniques, offer the most effective route for prevention or control of odorous emissions from these intermittent sources. The City of Calgary favours staged implementation of odour control works at this facility. The effect of cleansing headworks exhaust air relative to the total plant odorous emission will be evaluated following installation of odour control equipment. Should odours persist beyond the plant boundary following headworks improvements, then additional steps will undoubtedly be taken to further reduce odour emissions from other sources. Modification to operational procedure for the anaerobic digesters will be implemented as well, which will reduce the release of odours from this source considerably.

Final odour equipment selection for the headworks was based on a cost evaluation of incineration, wet scrubber, deep bed dry scrubber and a thin bed dry scrubber system. Odour control works for all alternatives were sized to accommodate a normal ventilation rate of 17,000 cfm and a high ventilation rate of 29,000 cfm.

A summary cost evaluation of capital, operation and maintenance costs is presented in Table 8. The least cost alternative in terms of a total annualized cost is the two unit deep bed dry scrubber system, as shown in Figure 3. The total annualized cost includes debt servicing and annual operating and maintenance costs. The initial capital outlay for this system is $504,000, with a total annual operation and maintenance cost of $156,000 per year.

Lifetime of media for the proposed deep bed system is estimated to be one year. The deep bed odour control equipment will be housed in a separate heated building. Installation of odour control works necessitates modifications to the existing air handling systems. These modifications are intended to increase the efficiency of the proposed odour control units by reducing the number of exhaust outlets and the amount of exhaust air.

Examination of the air balance report indicates that the building presently is operating under a significant negative pressure, resulting in a negative quantity of 2,983 cfm under a normal ventilation rate and 3,939 cfm under a high ventilation rate. These quantities can be substantially reduced without affecting system performance, but with a net effect of approximately 20% reduction in odour control equipment capacity. The number of foul air exhaust units were reduced from six to three. This was accomplished by modifying the air handling system for the air compressor room to use outside air instead of foul building air and by discharging foul grit pump room air at ceiling level on the main floor where it is diluted with grit chamber makeup air. In addition to the foregoing, one large exhaust fan was eliminated.

TABLE 8

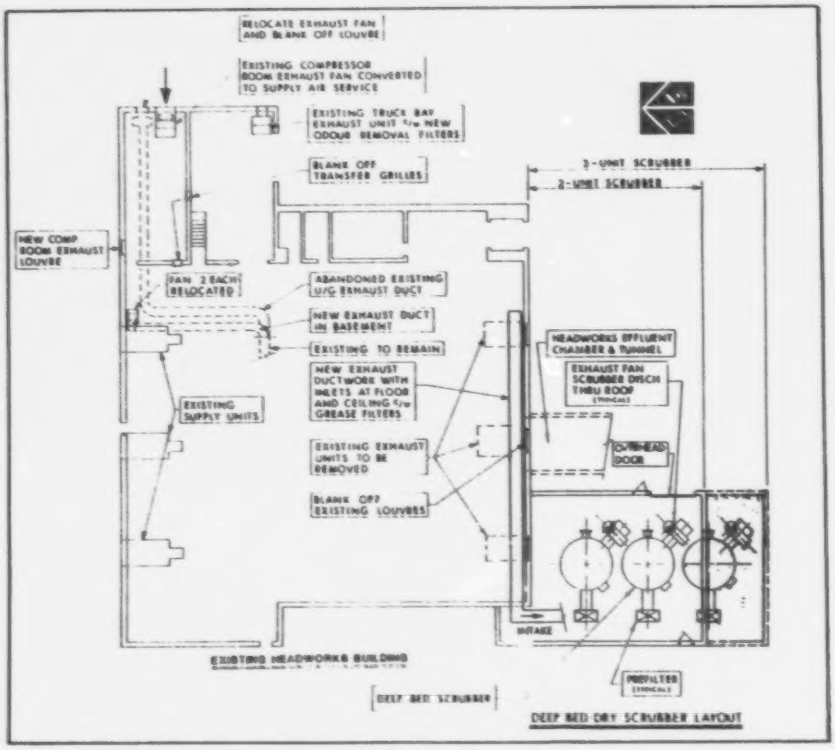
Summary Evaluation of Odour Control Alternatives

	Cost Estimates			
Control System	Capital	Annual Operating and Maintenance	Total Annualized Cost	Estimated Odour Control Efficiency (%)
Incineration @ 85% heat recovery	1,310,000.00	274,000.00	467,000.00	90
Incineration @ 60% heat recovery	686,000.00	461,000.00	562,000.00	90
Wet Scrubber System	719,000.00	222,000.00	328,000.00	70-90
Deep Bed Dry Scrubber System				
. 2 units	504,000.00	156,000.00	230,000.00	70-90
. 3 units	627,000.00	163,000.00	256,000.00	70-90
Thin Bed Dry Scrubber System	492,000.00	179,000.00	252,000.00	70-90

Ventilation for the grit chamber area is presently provided by three supply air units and three exhaust air units. During normal ventilation, two of the three systems operate with the third unit starting when high rate ventilation is required. All existing exhaust units, along the south wall, will be removed along with the ductwork. The existing exhaust louvres will be blanketed off with insulated panels. New air inlets will be located at the floor and ceiling levels along the south wall of the building and will be ducted to the two proposed deep bed odour removal units. Each inlet will be equipped with a prefilter designed to remove air borne grease and reduce buildup within the ductwork.

FIGURE 3

City of Calgary
Fish Creek Sewage Treatment Plant
Conceptual Layout for Deep Bed
Dry Scrubber System


RELOCATE EXHAUST FAN AND BLANK OFF LOUVRE
EXISTING COMPRESSOR ROOM EXHAUST FAN CONVERTED TO SUPPLY AIR SERVICE
EXISTING TRUCK BAY EXHAUST UNIT c/w NEW ODOUR REMOVAL FILTERS
BLANK OFF TRANSFER GRILLES
3-UNIT SCRUBBER
2-UNIT SCRUBBER
NEW COMP ROOM EXHAUST LOUVRE
FAN 2 EACH RELOCATED
ABANDONED EXISTING U/G EXHAUST DUCT
NEW EXHAUST DUCT IN BASEMENT
EXISTING TO REMAIN
NEW EXHAUST DUCTWORK WITH INLETS AT FLOOR AND CEILING c/w GREASE FILTERS
EXISTING SUPPLY UNITS
EXISTING EXHAUST UNITS TO BE REMOVED
BLANK OFF EXISTING LOUVRES
HEADWORKS EFFLUENT CHAMBER & TUNNEL
EXHAUST FAN SCRUBBER DISCH THRU ROOF (TYPICAL)
OVERHEAD DOOR
INTAKE
EXISTING HEADWORKS BUILDING
DEEP BED SCRUBBER
PREFILTER (TYPICAL)
DEEP BED DRY SCRUBBER LAYOUT

Because this system has a high pressure drop, the exhaust fans and ductwork must be sized accordingly. Preliminary sizing indicates that two new fans will be installed, each unit with a 75 hp motor. The fans will be installed in the odour control structure as shown on Figure 3. Clean air exhausted from each deep bed odour removal unit will be ducted through the roof from where it will be discharged vertically.

A thin bed dry scrubber will be used to treat the low volume exhaust from the truck bay only. This unit will be mounted in the headworks building in the location of the present exhaust fan for the truck bay.

The odour control system must be kept operational in case of a power failure and since the capacity of the existing standby power generator is presently fully utilized, a new 175 kW generator, complete with transfer switch and circuit breaker will be installed in the main control building in the area of the existing unit. A new feeder will be run to the existing control/MCC room in the headworks building where a new MCC will be installed along the west wall.

The City of Calgary have adopted the findings and recommendations contained in the preliminary design report and are currently in the process of implementing the detailed design stage for headworks odour control and some of the modifications outlined for the anaerobic digestion system. We anticipate construction to commence in October 1986.

THE BONNYBROOK WASTEWATER TREATMENT PLANT

by

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HISTORY

Calgary first began to treat its wastewater in 1933 when the original primary plant, with a capacity of 72,700 m^3/d (16 mgd), was built on its present site on the banks of the Bow River. Two square primary clarifiers which were part of that original plant remain in operation today. In 1958, the primary plant was expanded to give a total plant capacity of 213,000 m^3/d (47 mgd).

The first secondary treatment plant incorporating the activated sludge process with surface aeration was completed in 1970 together with a further expansion of the primary plant. The secondary plant had a capacity of 236,000 m^3/d (52 mgd) and the expanded primary plant, a capacity of 250,000 m^3/d (55 mgd).

The latest expansion which was commissioned in 1985 doubled the plant's capacity to 450,000 m^3/d (99 mgd) and for the first time tertiary treatment in the form of phosphorus removal was included. The expansion also included a power generation facility and a computer based data acquisition and control system.

In addition to the Bonnybrook plant, the City of Calgary is also served by the Fish Creek high purity oxygen activated sludge plant which has a capacity of 72,700 (16 mgd).

THE PROCESS FLOW STREAM

Figure 1 shows the layout of the Bonnybrook plant and Table 1 gives the sizes and capacities of the main treatment units.

The wastewater arrives at the plant in a gravity sewer with sufficient hydraulic head to pass through the treatment processes and discharge into the Bow River without the need to pump the main process flow stream.

Preliminary Treatment

In the headworks, the flow divides evenly between the new and old screen channels. The three old screens are of the curved bar type with a hydraulic rake, whereas the three new ones have inclined bars with a chain-driven climbing rake. There are two pairs of spiral flow aerated grit tanks each with a suction grit dredger and a grit classifier. Screenings and grit are discharged onto conveyors which in turn discharge directly into a bulk garbage container for removal to a City landfill site.

TABLE 1

BONNYBROOK WASTEWATER TREATMENT PLANT

DESIGN CRITERIA

Average Daily Flow	450,000m^3/d (99 mgd)
Peak Daily Flow	900,000m^3/d (196 mgd)

Headworks

No. of Screens = 6
Width of Screen = 1.8m
Bar Spacing = 25 mm

No. of Grit Tanks = 4
Size = 33.5X15.7X5.3m
Average Retention = 10.8 minutes

Primary Clarifiers = 12 No.

	No's.1 & 2	No's.3 & 4	No's.5 to 12
Size (m)	30.5X30.5X3m	30.5 diaX3.7m	33.0 diaX3.7m
Av. Overflow Rate m^3/m^2/d	39.7	39.7	41.3
Max.Overflow Rate m^3/m^2/d	79.5	79.3	80.6

Aeration Tank = 8 No.

	No's. 1 to 4	No's. 5 to 8
Size each tank (m)	83.9X3.4X4.6	66.5X14X6
Type & No. of aerators per tank	10 No. Surface Aerators	12 No. Jet Aerators
Aerator size (kw)	45	-
No. & size of air blowers (kw)	-	4 No. each 670 kw
No. & size of mixed liquor pumps	-	20 No. each 45 kw
Av. retention time (hrs)	6	7
Min. retention time (hr)	3.0	3.3

Secondary Clarifiers = 24 No. (Plus 4 future)

	No's. 1 to 12	No's. 13 to 24
Size (m)	32 X 32	39 dia
Av. Overflow Rate m3/m2/d	17	17
Max.Overflow Rate m^3/m^2/d	34	34

Sludge Digestion Tanks - 12 No. (10 Primary & 2 Secondary)

	No's.1 to 4	No's.5 to 8	No's.9 to 12
Size (m)	30.5X30.5X3m	30.5 diaX3.7m	35.0 diaX3.7m
Average Retention time (d)	34 days		(Secondary 4 days)
Organic loading kg VSS/m^3.d	1.0		(Secondary 2-3)

Primary Gravity Thickeners - 3 No.

Size (m)	15.2 dia X 3.0
Av. Overflow Rate m^3/m^2/d	24.4

Waste Activated Sludge Dissolved Air Flotation Thickeners = 4 No.

Size (m)	13.4X4.29X3.96
Average solids loading kg/m^2.d	240
Thickened sludge density (%)	4-6
Solids Removal Efficiency (%)	90-98

Power Generation & Heating

4 No. engine generators each 750 kw on digester gas 800 kw on natural gas.
3 No. 1957 kw boilers and 1 No. 2936 kw boiler
4 No. 1013 kw heat recovery units on the engine generators.
2 No. 300 kw natural gas engines

Phosphorus Removal

3 No. alum storage tanks each 5.9 m dia X 5.8 m

10 No. metering pumps 5 No. 835 l/hr
5 No. 1300 l/hr capacity

Figure 1

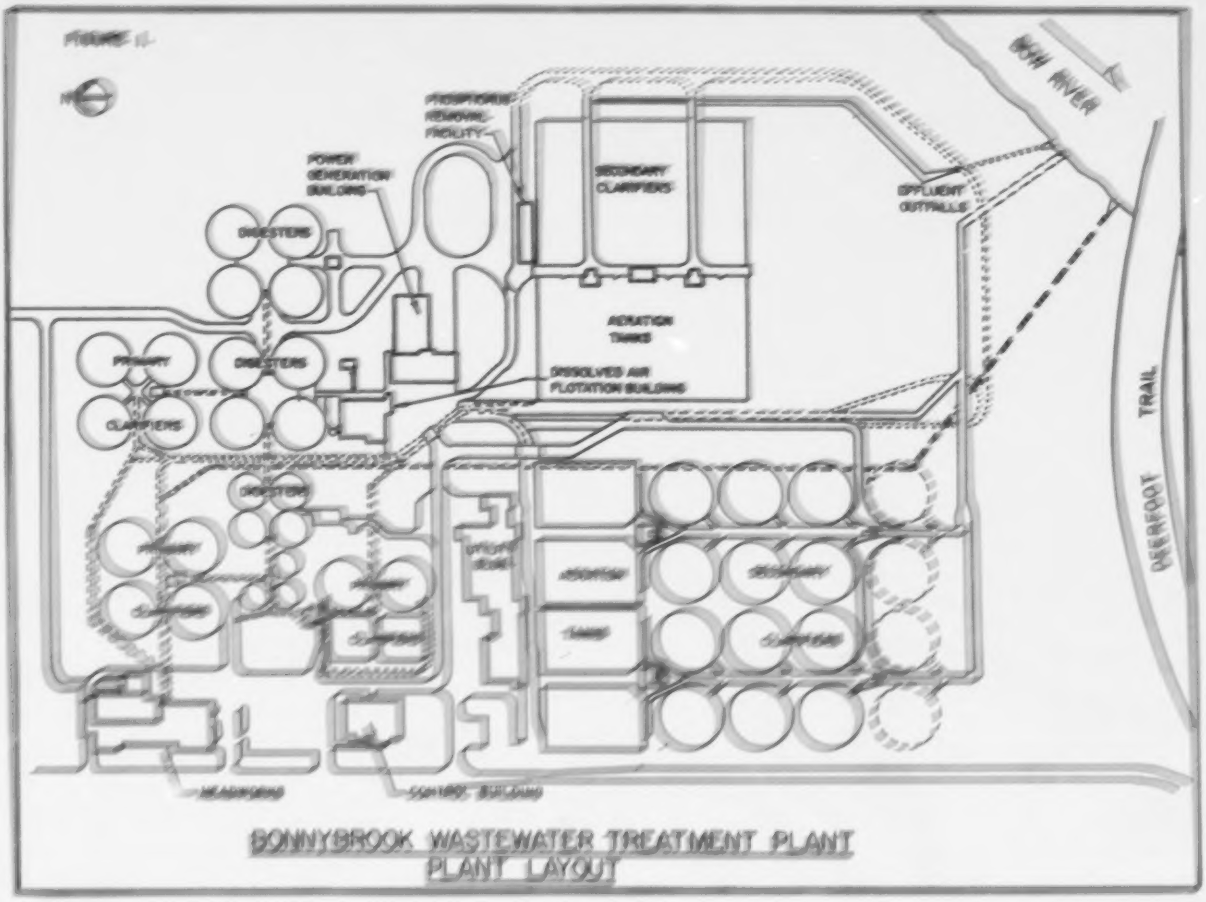


FIGURE 1
BOW RIVER
PHOSPHORUS REMOVAL FACILITY
POWER GENERATION BUILDING
SECONDARY CLARIFIERS
EFFLUENT OUTFALLS
DIGESTERS
AERATION TANKS
PRIMARY
DIGESTERS
DISSOLVED AIR FLOTATION BUILDING
CLARIFIERS
TRAIL
DIGESTERS
DEERFOOT
PRIMARY
AERATION
SECONDARY
CLARIFIERS
TANKS
CLARIFIERS
CLARIFIERS
HEADWORKS
CONTROL BUILDING
BONNYBROOK WASTEWATER TREATMENT PLANT
PLANT LAYOUT

Primary Treatment

There are three groups of four primary clarifiers. Apart from the two original square tanks, the clarifiers are all circular and all have mechanical scrapers and scum skimming devices. The scum is pumped directly to the digesters. The sludge is pumped to gravity thickeners from where the thickened sludge is pumped to the sludge blending tank and the decant is returned to the primary clarifiers.

Secondary Treatment

Effluent from all 12 primary clarifiers converges to one channel which subsequently divides to feed the new and the old air activated sludge secondary treatment facilities. The old secondary aeration tanks use fixed bridge surface aerators, whereas the new use the jet aeration system. Both the old and new secondary facilities consist of four aeration tanks. From each of the total number of eight aeration tanks the mixed liquor flows to three final clarifiers where the activated sludge is separated from the final effluent which flows out through the effluent channels to the Bow River. The activated sludge is returned to the head of the aeration tanks using screw pumps. Surplus or waste activated sludge is withdrawn from the return actirated sludge line and pumped to the dissolved air flotation thickeners. From here the thickened secondary sludge is pumped to the sludge blend tank for mixing with the gravity thickened primary sludge before being fed to the digesters.

Phosphorus Removal

Liquid alum is fed from the bulk storage tanks in the phosphorus removal facility to the aeration tanks where it reacts with the phosphorus in the mixed liquor to form aluminum phosphate which precipitates and settles out in the secondary clarifiers.

Sludge Digestion

Blended primary and secondary sludges are fed to the 12 digestion tanks which are heated to 35°C for mesophilic digestion. After an average retention of 34 days, the digested sludge, at an approximated 2% dry solids content, is discharged and pumped approximately 11 km to the Shepard sludge lagoons. In the lagoons the sludge settles to allow supernatant to be withdrawn and returned to the headworks of the Bonnybrook and Fish Creek plants. During the summer the thickened lagoon sludge, at approximately 11% dry solids content, is applied to agricultural land by tanker. The rate of application is 5.25 dry tonnes per acre.

Sludge Gas Utilization

Sludge gas is produced by the digesters at a rate of approximately 1200 m^3/hr. Its content is roughly 65% methane and 35% carbon dioxide. It is used to fuel gas engines which generate electricity and plant and process heat. If there is surplus gas it is used to fire the plant boilers. A pressurized gas storage system has been provided to enable surplus gas to be stored and utilized in the engines at the most advantageous times to reduce peak electricity demand charges.

Chlorination

The plant has two independent gaseous chlorination systems; one, which uses 150 lb cylinders, is effluent washwater disinfection and the other, which uses tonne containers, is for the control of return activate sludge bulking. The chlorination facility is housed in a specially designed separate building at the east end of the Utility Building.

In addition to the power generation facility, which is described in a separate paper at this conference, areas of the plant that are of particular interest include the jet aeration system, the computerised data acquisition and control system, and the phosphorus removal facility. These will now be described.

THE JET AERATION SYSTEM

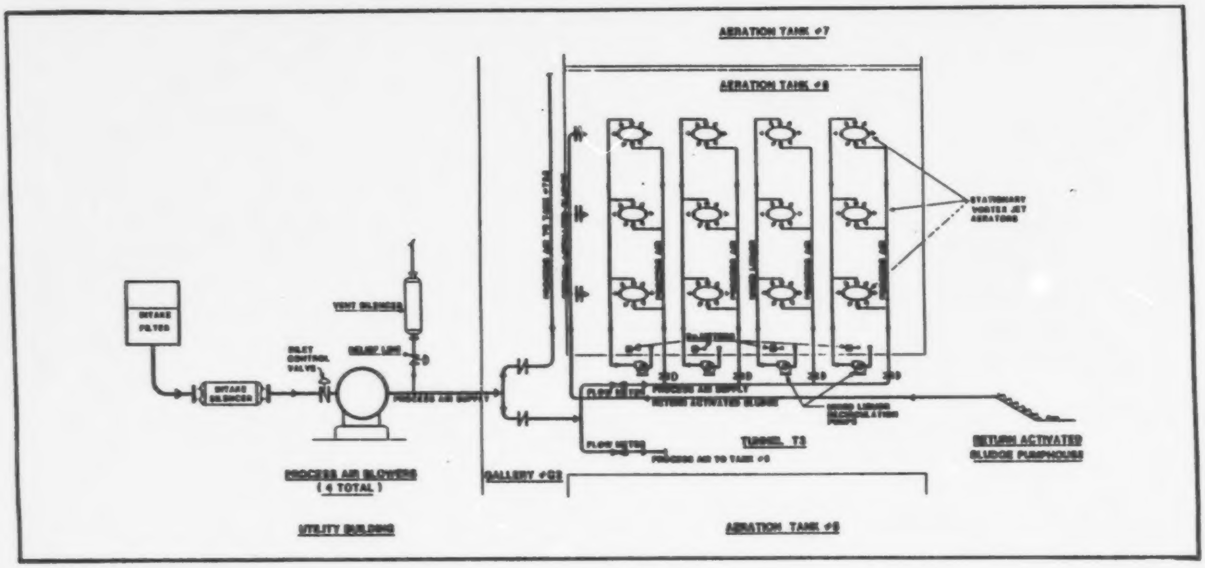
Process Design

The aeration equipment is designed to transfer 1.25 kg of oxygen per kg of BOD removed so as to maintain the BOD in the plant effluent below 15 mg/l. The total oxygen requirement at design flow is therefore 61,000 kg/d.

Aeration for the new secondary treatment process is achieved by diffusing air in the mixed liquor using a submerged jet aeration system. Recirculated mixed liquor from the aeration tanks is pumped to the jet aerators where it combines at the jets with the air supplied by the air blowers. The resulting mixture is discharged radially from the jet clusters across the aeration tank floor, resulting in fine air bubble diffusion. A schematic is shown in Figure 2.

Figure 2

Aeration System Schematic


AERATION TANK #7
AERATION TANK #8
STATIONARY VORTEX JET AERATORS
INTAKE FILTER
INTAKE SILENCER
INLET CONTROL VALVE
RELIEF LINE
VENT SILENCER
PROCESS AIR SUPPLY
PROCESS AIR BLOWERS
(4 TOTAL)
UTILITY BUILDING
GALLERY #G3
FLOW METER
PROCESS AIR SUPPLY
RETURN ACTIVATED SLUDGE
MIXED LIQUOR RECIRCULATION PUMPS
TUNNEL T3
FLOW METER
PROCESS AIR TO TANK #6
RETURN ACTIVATED SLUDGE PUMPHOUSE
AERATION TANK #5

The air is supplied by four identical centrifugal fixed speed blowers which discharge into a 1050 mm diameter manifold that feeds the air distribution piping system. One of the four blowers is standby to the other three. At present two blowers normally supply sufficient air. The blowers are located in the blower room at the west end of the Utility Building. The 1050 mm manifold divides into two 750 mm diameter headers each carrying the air supply for two of the four aeration tanks.

Each 750 mm diameter header further divides into two 450 mm diameter pipes, each of which carries the air supply to an individual aeration tank. Each of the 450 mm diameter pipes is equipped with an air flow meter to monitor the air flow rate to each aeration tank. All air piping within the tunnel and building area has been insulated to dampen the noise.

Within each aeration tank, four rows of jet clusters are fed in parallel from the 450 mm air pipe. Each row consists of three jet clusters equally spaced in the aeration tank to provide uniform distribution of air across the tank.

Four duty and one standby mixed liquor recirculation pump have been provided for each aeration tank. These mixed liquor recirculation pumps are located between the aeration tanks in the tunnel space. All the pumps have independent suctions and discharges with interconnecting pipework provided to allow the standby pump to be used in place of any one of the duty pumps.

Interconnecting pipework between the recirculation pumps of adjacent aeration tanks allows transfer of mixed liquor between each pair of tanks during tank dewatering.

Dissolved Oxygen Control

Each aeration tank is serviced by four independent sets of aerators. Each set includes one recirculating mixed liquor pump, three jet aerators, one air flow control valve, and one dissolved oxygen (DO) probe. The dissolved oxygen probes are suspended into the aeration tank from the walkway running between adjacent tanks.

Each aerator set can be operated either in the Automatic, Manual-Remote or Manual-Local mode as described in the following paragraphs:

Automatic Mode - Each dissolved oxygen probe controls the air supply to the adjacent set of three jet clusters by modulating the air flow control valve associated with it to maintain the dissolved oxygen concentration at an adjustable set point in the range of 0.0 to 5.0 mg/L. This set point can be adjusted by the operator in the Utility Building Control Room. At present the operators aim is to maintain a level of 1.5 mg/L on the inlet probe and 2 mg/L on the other three probes. If the dissolved oxygen concentration is less than an adjustable low alarm setpoint on the 0.0 to 5.0 mg/L scale, an alarm is initiated to warn the operator. This is normally set at 0.5 mg/L.

Manual-Remote Mode - In this mode each air control valve is manually operated from the keyboard in the Control Room in the Utility Building.

Manual-Local Mode - Each air flow control valve is manually operated from a locally mounted manual loading station in tunnels between the aeration tanks.

Air Blower Control

The four air blowers operate with identical characteristics in a parallel configuration. Air output from each blower is controlled with a blower inlet control valve. Any combination of the four blowers can be placed in duty service as the need arises.

Increasing Air Demand - If the air demand exceeds the maximum output for one operating blower in any mode, a pressure transmitter registers a low pressure warning in the Area 6 Control Room in the Utility Building. A second blower must then be started by the operator. Similarly, if the air demand exceeds the combined maximum output for two operating blowers, the pressure transmitter again registers a low pressure warning in the Control Room and a third blower must then be started by the operator.

Decreasing Air Demand - If the air demand is less than the combined output for three blowers in any mode, a surge pre-alarm registers a warning in the Control Room. This indicates to the operator that the alarming blower be stopped immediately.

If the air demand is less than the combined output for two blowers, a surge pre-alarm is again registered and the alarming blower must be stopped by the operator.

Each blower motor must be independently started with a local motor starter in the Manual-Local mode. Inlet control valves are operated either in the Automatic, Manual-Remote or Manual-Local mode.

The control modes are as follows:

Automatic Mode - All inlet valves respond to a common input signal generated by the pressure transmitter on the main air header which has an operating range of 55 to 76 kPa and an adjustable setpoint. When the air system pressure falls below the setpoint, all operating inlet control valves simultaneously modulate open. When the air system pressure exceeds the setpoint, all operating inlet control valves simultaneously modulate to throttle the inlet.

Any combination of two, three or four inlet control valves can simultaneously operate in the automatic mode. If three or fewer valves are operated in the Automatic mode, the remaining valves either operate in the Manual-Local mode or are closed.

Manual Remote Mode - In this mode, each control valve is operated from the keyboard in the Control Room.

Manual Local Mode - The intake control valve for each blower can be operated manually from a local manual loading station.

THE COMPUTER DATA ACQUISITION AND CONTROL SYSTEM

The computer data acquisition and control system, CDACS, is a distributed process control and data monitoring system. The system has over 400 analog points and approximately 1500 digital points. Figure 3 shows a geographical layout of the system. Because it is a distributed architecture, each area of the system functions independently. Data is collected and stored by the Central Processing Unit which has the capability to perform total plant control. Control can be carried out at three levels; locally at the equipment, remotely from the area control console, or centrally from the central control consoles.

The plant is divided into six areas for the purposes of monitoring and process control:

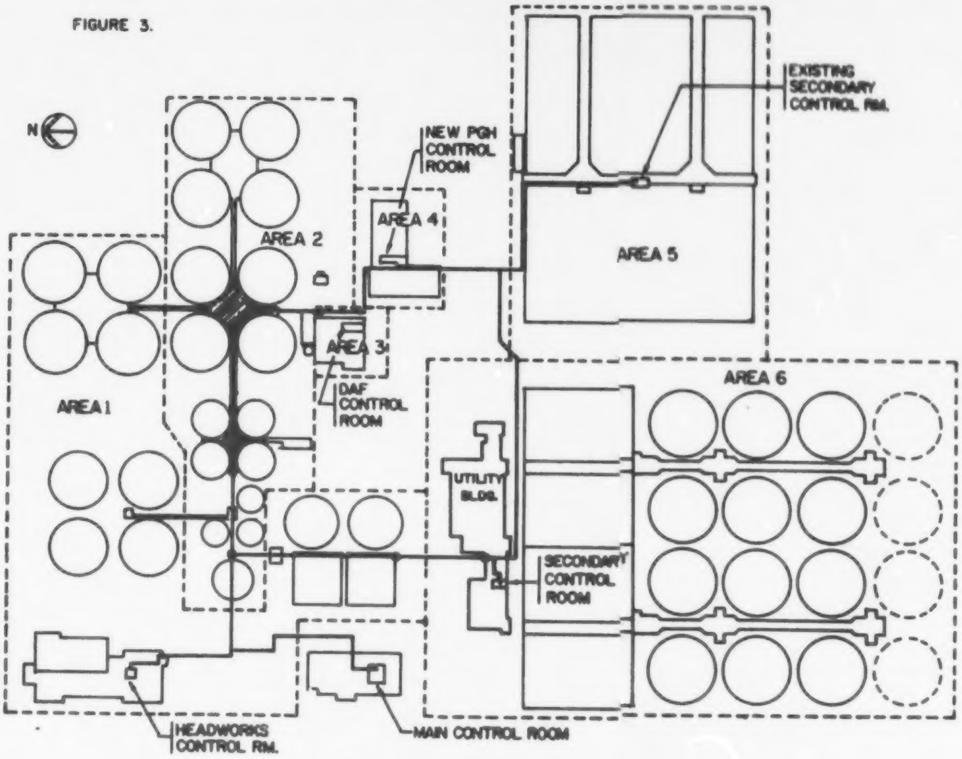
Area 1 Primary Treatment
Area 2 Digestion Process
Area 3 Dissolved Air Flotation Building
Area 4 Power Generation and Heating Plant
Area 5 Secondary Process Aeration Tanks 1 to 4
Area 6 Secondary Process Aeration Tanks 5 to 8

Each area has its own control room with a local control console. Under the present arrangement the treatment processes are monitored and controlled by the plant operators at the six areas.

From the central control room, where the central processor unit and the central monitoring console are housed, the Chief Operator can monitor the entire treatment plant. In addition, in the Area 2 Digester Control Room, and the Area 6 Secondary Process Control Room, there are distributed monitoring consoles which have identical characteristics to the central control console. They are used by the area operators to monitor the process in Areas 1, 2, 3, and 4 and Areas 5 and 6 respectively.

Figure 3

FIGURE 3.


N
NEW PGH CONTROL ROOM
EXISTING SECONDARY CONTROL RM.
AREA 2
AREA 4
AREA 5
AREA 3
DAF CONTROL ROOM
AREA 1
AREA 6
UTILITY BLDG.
SECONDARY CONTROL ROOM
HEADWORKS CONTROL RM.
MAIN CONTROL ROOM

CDACS GEOGRAPHIC LAYOUT

The central processor's main function is for data acquisition. Here, process data is collected from the field, laboratory data is input, calculations are performed and logs are generated.

At the central control console and the Area 6 and Area 2 control consoles a variety of displays can be called up on the CRT screens. These displays include:

- Trend displays which allow the monitoring of up to four process variables at any one time. Trend displays can be historical or current and can be stored for future reference.

- Plant area dynamic graphics, giving current status for all process areas showing which equipment is running and indicating all process variables, such as flow rates, dissolved oxygen levels, sludge densities, etc.

- Plant status displays, which provide an overview of the process in terms of alarm status.

- Plant deviation displays, which provide an overview of the process in terms of deviation from set points.

From all control consoles it is possible to carry out control functions, including the following:

- set point adjustment
- equipment start/stop
- valve open/close
- plant sequence change
- control parameter change
- alarm set point adjustment
- alarm acknowledge
- alarm silence

etc.

All control consoles are provided with a key lock switch to prevent certain functions such as control loop tuning parameters and control characteristics from being changed.

Examples of automatic control functions that CDACS can carry out include dissolved oxygen control, flow splitting between units, the rate activated sludge return and wasting and chemical dosage rates, etc.

THE PHOSPHORUS REMOVAL FACILITY

General

The facility is designed to reduce the concentration of total phosphorus in the plant effluent to an average of 1 mg/L as Phosphorus (P). The phosphorus concentration in the raw untreated sewage is approximately 7 mg/L and would be reduced to approximately 3 mg/L by the normal secondary treatment processes. To regularly achieve less than 1 mg/L requires additional or tertiary treatment.

Process Theory

At Bonnybrook, the further removal is achieved by physical-chemical means. The principal processes involved are precipitation, coagulation and adsorption. They occur at different stages in the treatment process.

Firstly, when a chemical which is a source of metal cations is added to sewage, any orthophosphate present will react and produce a fine colloidal phosphate precipitate. Simultaneously, the chemical reacts with natural alkalinity of the sewage to coagulate and flocculate any suspended material into a large floc. This floc then forms part of the sludge blanket created in the secondary clarifiers. The sludge blanket entraps the fine phosphate precipitate enabling it to settle out in the clarifier and be removed with the secondary activated sludge.

The most common chemicals that are used for this process are alum (aluminum sulphate) and ferric chloride. The Bonnybrook facility is designed to use alum in solution.

FACILITIES DESIGN

The phosphorus removal process consists of dosing the aeration tank effluent with chemicals upstream of the secondary clarifiers. The facilities required to do this can be divided into two elements:

1) Chemical Storage Facilities
2) Chemical Distribution Network

Both of these have to be sized in accordance with thw required chemical dosage. Jar tests at Bonnybrook indicated a dosage of between 60 mg/l and 80 mg/l would be required, and the facility has been designed to dose a maximum of 80 mg/l of alum ($Al_2 (SO_4)_3 14 H_2O$).

Chemical Storage Facilities

The storage facility has been designed for 10 days storage with a dosage of 80 mg/l for the 1993 design flow of 450,000 m^3/d. Because initially the flows are less than this, only three of the four storage tanks have been provided. The storage tanks each have an effective capacity of 440 m^3 and are wood stave tanks with PVC liners. The liquid alum is delivered in tankers which are unloaded using a compressed air system.

Chemical Distribution System

Liquid alum is fed through the PVC distribution pipework to the outlet channel of each aeration tank using 10 metering pumps. One pump is assigned to one aeration tank with 2 standby pumps. The pumps can be operated in one of three modes, automatic, manual-remote and manual-local. In the automatic mode, the dosing rate is paced to the incoming flow to each aeration tank using an algorithm in the CDACS system. In the manual remote and manual modes the dosing pumps can be controlled from the secondary treatment plant control rooms and locally at the pumps. In the manual modes the operator has to vary the pumping rate according to plant flow.

PLANT PERFORMANCE

The plant was designed to meet Alberta Environment standards for the discharge of treated wastewater to the Bow River. These are as follows:

The monthly average BOD must be less than 20 mg/l.
The monthly average SS must be less than 20 mg/l.
The plant must reduce the phosphorus concentration to 1 mg/l or less between May and October and 1.25 mg/l or less between November and April.

The process design parameters for the overall plant are given in Table 2 following:

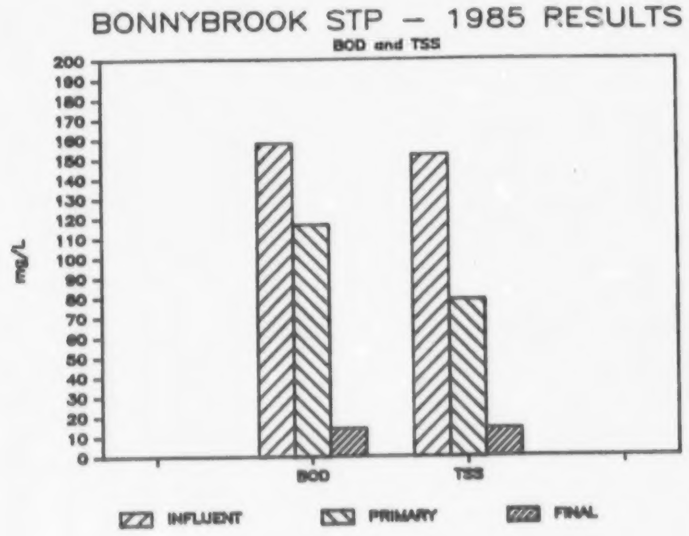
Bonnybrook Wastewater Treatment Plant
Design Parameters

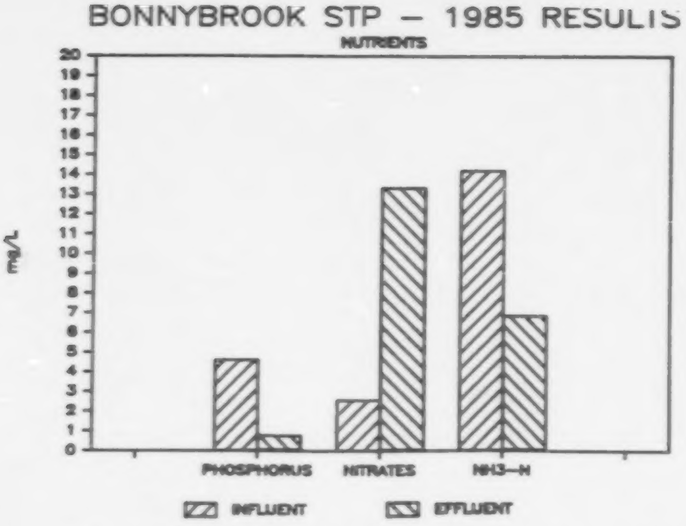
Parameter	Average
Flow Rate, m^3/d	450,000 (99 mgd)
Raw Wastewater BOD, mg/l	146
kg/d	65,600
Raw Wastewater SS, mg/l	202
kg/d	90,900
Primary Effluent BOD, mg/l	117
kg/d	52,500
% Removal	20
Primary Effluent SS, mg/l	100
kg/d	45,400
% Removal	50
Secondary Effluent BOD, mg/l	15
kg/d	6,750
% Removal	90
Secondary Effluent SS, mg/l	15
kg/d	6,750
% Removal	93
Secondary Effluent P, mg/l	1.0
kg/d	450
% Removal	80

Figures 4, 5, 6 and 7 show the plant performance in terms of effluent quality as compared to influent and primary effluent. Figure 4 shows BOD and SS removals, Figure 5 nutrient removals and Figures 6 and 7 metals.

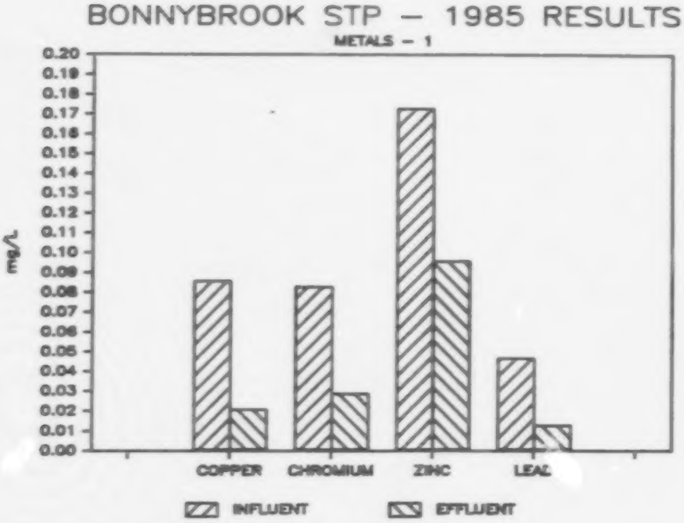
From Figure 5 it is apparent that some nitrification is taking place.

From Figures 6 and 7 it is apparent that, with the exception of nickel, there is a significant reduction in metals through the plant.

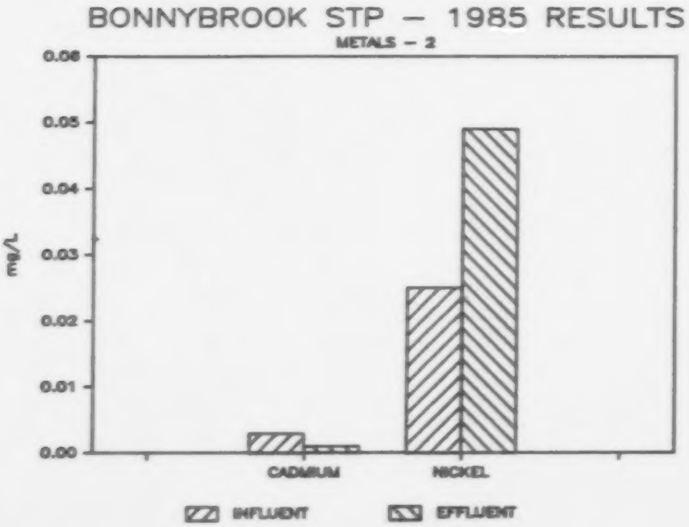



BONNYBROOK STP – 1985 RESULTS

NUTRIENTS


BONNYBROOK STP – 1985 RESULTS

METALS – 1


BONNYBROOK STP – 1985 RESULTS

METALS – 2

DESIGN AND CONSTRUCTION

Reid Crowther and Partners Ltd. were the prime consultants for the design of the latest expansion of the Bonnybrook Wastewater Treatment Facility. UMA Engineer Ltd. were the associate consultants responsible for part of the work.

Because of the magnitude of the expansion, eight separate contracts were awarded as follows:-

Sludge Treatment Complex

4 digesters, digester pumphouse and associated equipment and instrumentation
Contractor: PCL Construction Ltd.
Cost: $19,386,952
Awarded: May 13, 1982
Substantially Complete: May 2, 1985

Secondary Treatment Facilities

4 aeration tanks, 12 secondary clarifiers, utility building, effluent outfall and associated equipment and instrumentation
Contractor: Pigott Construction [Western] Ltd.
Cost: $49,089,000
Awarded: August 18, 1982
Substantially Complete: May 7, 1985

Primary Treatment and Headworks

Headworks building, 2 aerated grit tanks, 4 primary clarifiers, annex to control building and associated channels, equipment & instrumentation
Contractor: Ellis-Don Ltd.
Cost: $8,543,000
Awarded: August 18, 1983
Substantially Complete: September 10, 1985

Power Generation and Heating

4 engine generators, 4 heat recovery units, 2 digester gas storage tanks, gas storage compressor, electrical controls and instrumentation and building
Contractor: W.A. Stephenson [Western] Ltd.
Cost: $8,055,796
Awarded: January 4, 1983
Substantially Complete: October 7, 1985

Phosphorus Removal Facility

Chemical storage vessels, metering pumps, distribution piping, and building
Contractor: P & M Construction Ltd.
Cost: $1,969,000
Awarded: January 15, 1982
Substantially Complete: December 13, 1982

Emergency Relief Sewer and Siteworks

Emergency relief sewer, final site grading, landscaping and trees
Contractor: Volker Stevin Contracting Ltd.
Cost: $1,629,906
Awarded: September 12, 1985
Completed: May 30, 1986

Instrumentation

Upgrading existing instrumentation
Contractor: Black & McDonald Ltd.
Cost: $901,340
Awarded: December 21, 1984
Substantially complete: November 1, 1985

Digital Instrumentation

Supply and installation of all computer hardware and software
Contractor: Fischer & Porter (Canada) Ltd.
Cost: $1,146,101
Awarded: August 16, 1983
Substantially complete: January 30, 1986

FULL-SCALE OPERATION OF GRANULAR ACTIVATED CARBON CONTACTORS AT THE BUFFALO POUND WATER PLANT

by

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INTRODUCTION

The major source of water for Regina (pop.170,000) and Moose Jaw (pop.35,000), is Buffalo Pound Lake, a 30km (18 mile) long and 1km (0.6 mile) wide artificial reservoir with a mean depth of 3m (10 ft). Extensive blooms of blue-green algae (anabaena, aphanizomenon and microcystis) occur in summer[1] when the temperature of the water reaches 20-25°C. These algae blooms cause a number of problems, primarily in July and August, the most evident to the consumer being the objectionable taste and odour, which has been described as sour, musty, grassy, septic, pigpen etc. Associated problems include high total organic carbon (TOC), high chlorine demands (3-12 mg/L), interference with alum coagulation, excessive trihalomethane (TTHM) formation, algae toxins[2], many organic compounds[3], and large numbers of algae cells. Trihalomethane levels do not exceed the Canadian Federal[4] and Provincial[5] limits of 350 ug/L but occasionally exceed 100 ug/l which is the U.S. limit[6].

This paper addresses the design, construction and operation of the granular activated carbon (GAC) contactor system (including on-site carbon regeneration) chosen to reduce taste and odour, trihalomethanes and total organic carbon.

Background

Plant staff started looking at granular activated carbon in 1976 and after initial pilot scale columns gave promising results, experimented with exchanging the top 0.60m (2 ft) of filter media on one full-scale filter from 1977 to 1979. Some benefits were obtained (odour, TOC, and TTHM reduction) but there were serious drawbacks including poor turbidity removal, large carbon losses on backwash, and limited capacity[7].

A consultant (CH2M Hill) was employed in 1980 to evaluate various options including alternative intake sites, different oxidants and coagulants, powdered activated carbon, dissolved air flotation and GAC. Conclusions reached at the end of the study[8] were that downflow GAC contactors in the post-filtration mode following conventional treatment gave the best reductions in odour, TOC and TTHMs.

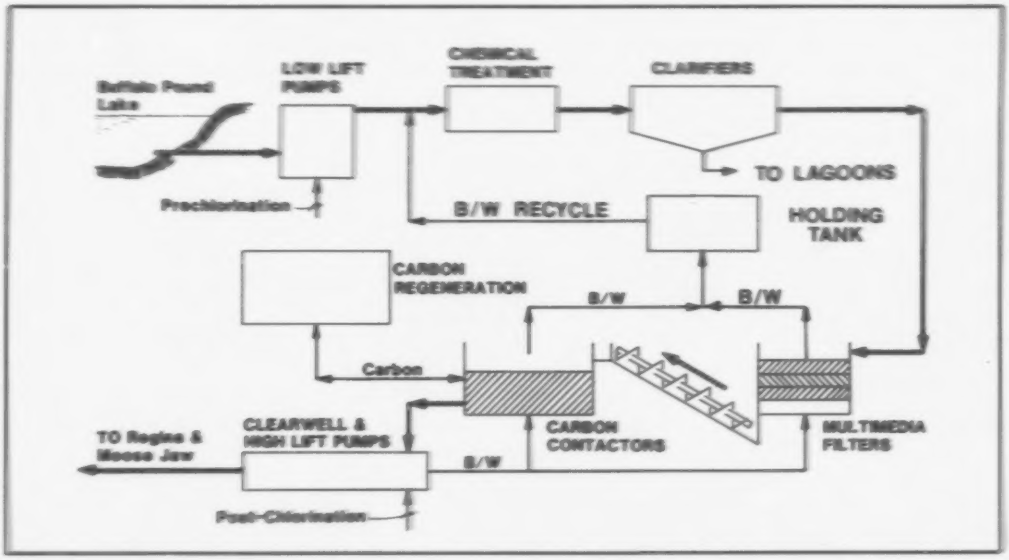
Early in 1981 CH2M Hill was authorized to proceed with design of the full-scale system which included the carbon contactors, a carbon regeneration plant and a lagoon/holding tank waste management system. The project was delayed however, to allow the Provincial Government to gather information it required to formulate long-term water policies for the region before committing funds to the GAC project. A number of studies^{9,10} were commissioned and carried out in 1981 and 1982 to look at water supply alternatives and the conclusions of these studies were that although there were a number of viable long-term alternatives, the GAC system could be constructed at any time without restricting future options.

Late in 1983 funding arrangements were agreed upon and final design and tendering of contracts resumed in January of 1984. Construction started in May 1984 with the GAC contactor system^{11} being commissioned in June 1985 and the carbon regeneration system in January 1986.

FACILITY DESIGN

Before the GAC addition, the treatment plant was of conventional design [Figure 1] with pre-chlorination, alum coagulation/flocculation with polymer aids, settling in upflow reactor-type clarifiers equipped with 60 degree tube settlers, pH correction, and mixed-media filtration. Plant flows were 54-136 dam^3/day [12-30 MIGD] and water was delivered to the cities from the plant clearwell through a 58km[36 mile] pipeline to Regina and a 20km[12mile] pipeline to Moose Jaw.

Figure 1 - Plant Schematic


Buffalo Pound Lake
LOW LIFT PUMPS
CHEMICAL TREATMENT
CLARIFIERS
TO LAGOONS
Prechlorination
B/W RECYCLE
HOLDING TANK
CARBON REGENERATION
B/W
B/W
Carbon
CARBON CONTACTORS
MULTIMEDIA FILTERS
TO Regina & Moose Jaw
CLEARWELL & HIGH LIFT PUMPS
B/W
Post-Chlorination

Under average summer conditions the plant normally produced a finished water with low turbidity [< 0.2 NTU], and colour [< 5 Pt/Co units], but with high threshold odour numbers [5-15], high total organic carbon [3-6mg/L C] and appreciable trihalomethanes [70-150 ug/L].

GAC Contactors

The new GAC contactors were basically of concrete filter box design [Figure 2] with 92.9 m^2 [1000 ft^2] of surface area, 3m [10 ft] depth of carbon and another 2.1m [7 ft] of water depth from the carbon to the backwash troughs. The underdrains consisted of plastic pipe laterals wound with 0.5 mm wedge wire. From the contactors the water passes into a new clearwell area where it is post-chlorinated for distribution system disinfection. This design [Figure 1] required elevating the water approximately 5m [16.4 ft] to the inlet channel to the contactors using Archimedes screw pumps [diameter 2.44 m [8 ft], motor size 250 HP]. The 8 beds each contain 172 m^3 [6074 ft^3] of carbon and are designed for a minimum empty bed contact time [EBCT] of 15 min [Table 1]. Total amount of carbon in use is 689,000 kg [1,520,000 lbs].

Figure 2 - Contactor detail

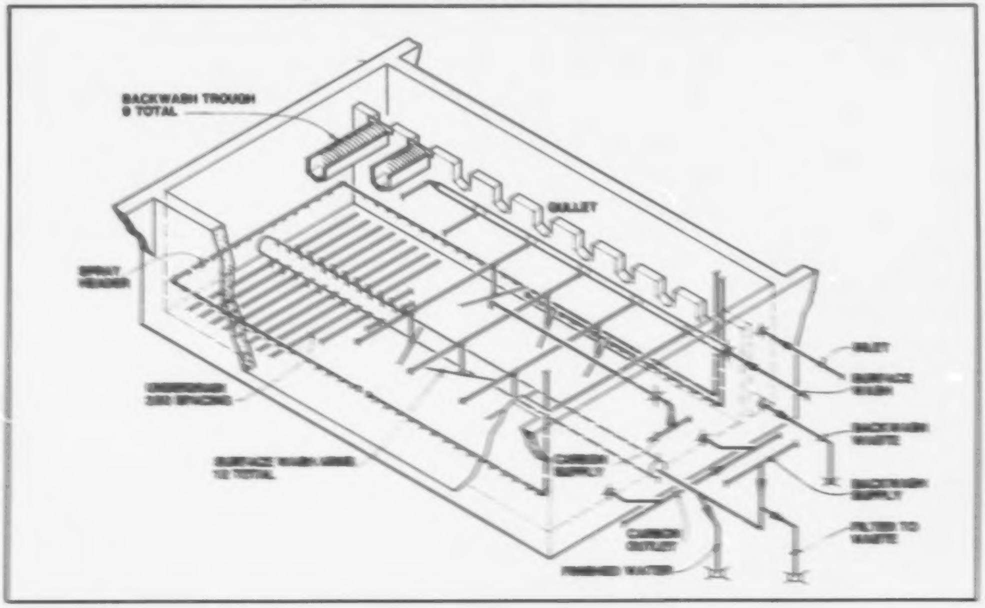

BACKWASH TROUGH
8 TOTAL
GULLET
SPRAY HEADER
UNDERDRAIN
SURFACE WASH ARMS
12 TOTAL
CARBON SUPPLY
CARBON OUTLET
FINISHED WATER
INLET
SURFACE WASH
BACKWASH WASTE
BACKWASH SUPPLY
FILTER TO WASTE

Table 1 - Carbon Bed Parameters

Parameter	Imperial Units	Metric Units
Mode	**(Post-filtration rapid gravity contactors)**	
Number of Beds	**8**	**8**
Volume of Carbon/Bed	**6074 ft³**	**172 m³**
Weight of Carbon/Bed	**190,000 lb**	**86,000 kg**
Total Weight Carbon	**1,520,000 lb**	**689,000 kg**
Bed Depth	**10 ft.**	**3.05 m**
Surface Area	**630 ft²**	**58.5 sq. m**
Design Flow	**4.16 gpm/ft²**	**12.2 m/hr**
Flow Range (Actual)	**1.5-4 gpm/ft²**	**4.4-11.7 m/hr**
Empty Bed Contact Time at 2 gpm/sq. ft.	**31.1 min**	**31.1 min**
Empty Bed Contact Time at 4 gpm/sq. ft.	**15.6 min**	**15.6 min**

One of the contactors was equipped with stainless steel sample ports at 30 cm [1 foot] intervals down the side wall over the depth of the carbon. These ports were screened and extended 45 cm [17.7 in] into the carbon.

Carbon Transfer System

Spent carbon is transferred to the regeneration plant by a pressurized slurry system along a 60m [200 ft] access tunnel. The carbon is gravity slurried from the bottom of the contactors into a 14 m^3[500 ft^3] blowcase, then pressurized with motive water pumps to 350 Kpa [50 psi], and then transferred either to the furnace day feed tank or to regeneration plant storage tanks. Once regenerated the carbon is fed into one of two 7 m^3 [250 ft^3] blowcase tanks to be slurried either to storage or back to the contactors. An alternative transfer method is to use high pressure pumps and eductors.

Carbon regeneration

The cost of carbon was $1.72/kg [$0.78/lb] delivered and placed while the cost of regenerating carbon was estimated to be 0.33/kg [0.15/lb] including 10% losses. An onsite regeneration facility was therefore calculated to be economically justifiable. The cost advantage is sufficient to pay out the $3.5 million for the regenerator facilities in 4 years over a fill-and-dump operations and in approximately 7 years over regeneration at distant U.S. locations. Anticipated regeneration frequency is one per year.

The furnace is a Westvaco two-stage, gas-fired, fluid bed system[12]. The furnace capacity of 6350 kg/day [14,000 lbs/day] was deliberately oversized to allow for future water plant expansion plus the possibility of more frequent than anticipated regeneration. It is capable of regenerating the entire present inventory in 3.5 to 4 months [1 contactor every 13-16 days].

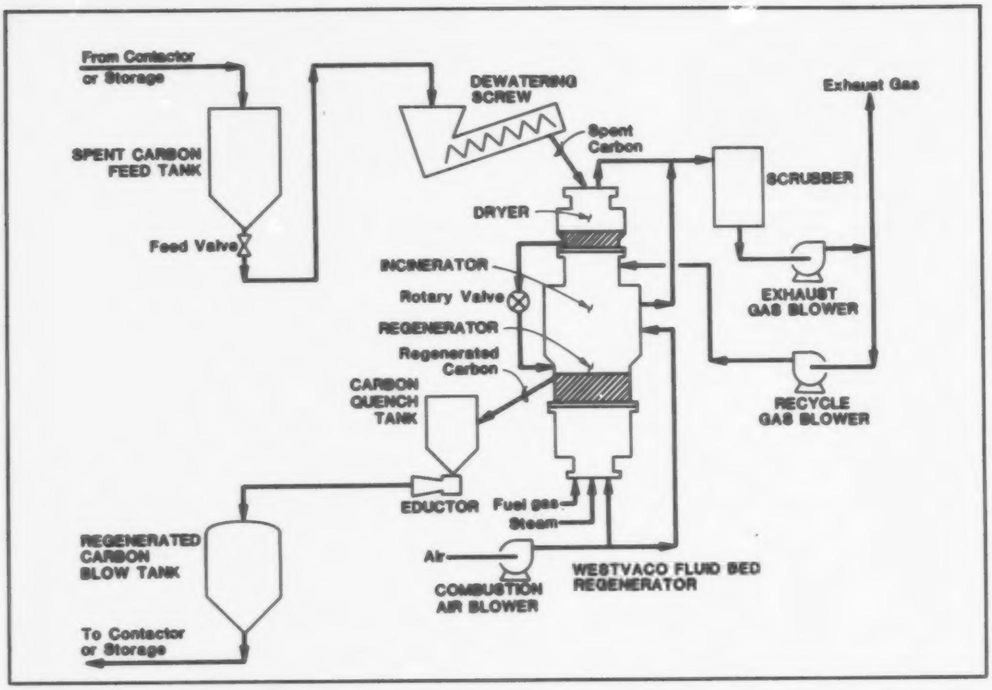
The activated carbon is regenerated thermally [Figure 3]. It is first dried at 135°C [275°F] and then heated to 850°C [1550°F] in an atmosphere of burner gas and steam with oxygen controlled at 0.2% to limit carbon combustion. The adsorbed organics are pyrolyzed at that temperature and the heat and steam combine to reactivate the adsorption sites. The pyrolyzed organics are incinerated in an atmosphere of excess oxygen and the regenerated carbon exits the furnace to a water quench tank and then to the return blowcases. The exhaust gases are sent through a two-stage wet scrubber to remove carbon fines and reduce gas temperature and moisture content. Hot gases from the incinerator section are used to dry the incoming wet carbon.

Carbon Specifications and Installation

Pilot plant testing of various carbons in 15 cm [6 in] diameter columns with 1.5m [5 ft] depths from 1981 to 1984 provided information on the effectiveness of carbon in removing different contaminants and allowed observation of operational problems[13,14]. The results of these tests were used to set specifications for the initial carbon contactor charge.

The results indicated a preference for hard, bituminous coal-based carbons with iodine numbers greater than 900 and CWS hardness greater than 75. Mesh size chosen was 8 x 30. Two different carbons were chosen for the initial charge after tenders were submitted; CECA Cecarbon 830 and Calgon Filtrasorb 300. It is intended that full-scale evaluations of carbon effectiveness and durability will be an on-going process.

Figure 3 - Furnace Design


From Contactor or Storage
SPENT CARBON FEED TANK
Feed Valve
DEWATERING SCREW
Spent Carbon
DRYER
INCINERATOR
Rotary Valve
REGENERATOR
Regenerated Carbon
CARBON QUENCH TANK
EDUCTOR
Fuel gas
Steam
Air
COMBUSTION AIR BLOWER
WESTVACO FLUID BED REGENERATOR
SCRUBBER
Exhaust Gas
EXHAUST GAS BLOWER
RECYCLE GAS BLOWER
REGENERATED CARBON BLOW TANK
To Contactor or Storage

The carbon was loaded into the contactors during April 1985 using water eductors from both bulk tanks (Calgon) and 1000-lb tote bags (CECA). The contactors were backwashed to level the carbon after the addition of each 1-foot depth and the top 1-2 inches of fines were scalped off after the beds were full. Before use the carbon was sterilized by the addition of 25% caustic soda to bring the pH above 12. The beds sat overnight and the caustic was backwashed out the following day.

RESULTS

The GAC contactors were operated for 135 days in 1985; this consisted of a 10-day commissioning period in June (10th-19th) and then continuous operation from July 3rd to November 4th. The carbon successfully removed odour and reduced TOC and TTHMs over the 4.5 month period. By November, odour had broken through at low levels and TOC and TTHM removals were down to 30-40%. Effective total exhaustion (effluent consistently at 95% or more of influent) was extrapolated at around 170-200 days. Table 2 lists raw water quality parameters for the operating period. The carbon was removed from the contactors and was regenerated from Jan. 10th to May 2nd, 1986.

The contactors were monitored continuously for operational problems, flow, turbidity, and headloss, daily for pH, chlorine residuals and standard plate counts, and weekly or better for TOC, trihalomethanes, odour, particle counts, geosmin, phosphate and algae cells. In addition, raw and finished water samples were monitored regularly for all major ions, trace nutrients, bacteria, metals, organics and physical parameters. Wavefront progressions for odour, TOC and TTHMs were followed by using the 0.3m (1 ft) interval ports. The contactors were each backwashed at 8-day intervals primarily to control bacteria levels.

Table 2 - Plant Parameters

Raw Water Data (Monthly Means)

Parameters	Units	June	July	August	September	October
Sum of Ions (calc)	mg/L	358	373	383	389	392
Total Alkalinity	mg/L CaC03	161	162	162	164	163
Total Hardness	mg/L CaC03	170	172	171	179	182
pH	pH units	8.1	8.2	8.3	8.1	8.3
Temperature	deg C	15	20	18	12	6
Turbidity	NTU	10	3.2	3.2	3.5	2.6
Chlorophyll a	ug/L	7	15	18	9	6
Organic Carbon (dissolved)	mg/L C	5.4	5.6	5.9	6.1	5.4
Threshold Odour Number	T.O.N.	13	24	24	23	28

Analytical Methodology

Turbidities were measured on a Turner model 40 nephelometer, free and combined chlorine residuals on a Wallace and Tiernan Amperometric titrator, pH on Orion 601 and 701 pH meters, and particle counts (2-120um) on a HIAC PC-320 Particle Counter. Standard Plate Counts, coliform bacteria, organic nitrogen, threshold odour numbers (TON), phosphate levels and algae cell numbers were measured using Standard Methods analyses[15]. Odour numbers were qualified by adding one or two adjectives to describe the odour.

Trihalomethanes were monitored on a gas chromatograph (Hewlett-Packard 5840A) equipped with an electron capture detector. A 5-ft Chromosorb 101 packed column (0.25" diam.) was used with direct aqueous injection (5ul samples) to separate and quantify the trihalomethanes[16].

Geosmin analyses were carried out on a capillary column (30m DB-1) gas chromatograph (Hewlett-Packard 5730A) equipped with a flame ionization detector. A 2-litre sample was extracted with two 5ml aliquots of pentane and concentrated to 0.5 ml for injection. This method[17] was used to monitor contaminants in the C6 to C35 range, but was primarily set up to look for geosmin and methylisoborneol.

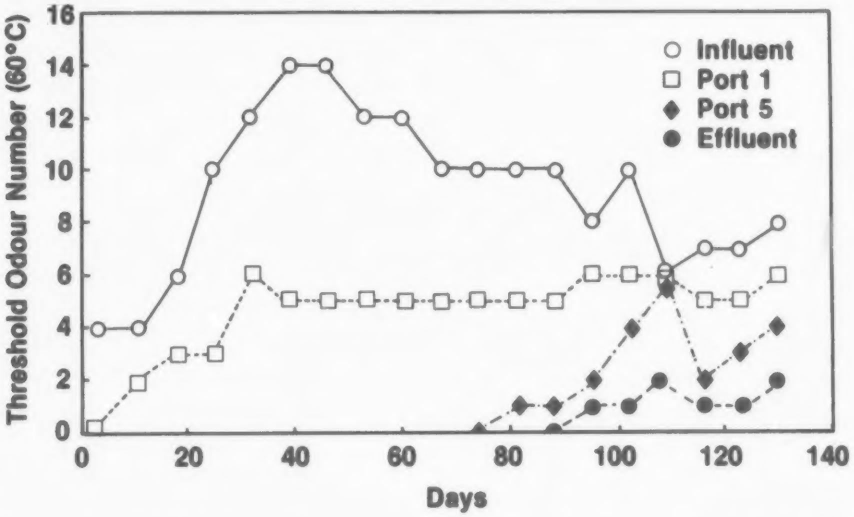
Total organic carbon was monitored either by carbon analyzer (Beckman 915-B IR model) or by UV absorption at 254 nm on a 2-nm bandwidth spectrophotometer using 10-cm cells. The precision of the UV method (+/- 0.1 mg/L) was better than the IR Analyzer (+/- 0.5 mg/L) but the UV method gave slightly lower results (by 0.5 mg/L) than the IR method and could only be used for filtered samples (DOC).

Odour

Raw lake-water odours of 10 to 30 TON (musty,sour) were reduced to 4 to 14 by conventional treatment and the GAC then eliminated this odour for 80 days

(Figure 4) after which low-level odours of 1 to 2 TON units of a slightly musty odour were found in the effluents. Odour breakthrough at the 1.52m (5-ft) level occurred at 75 days, so the bottom 1.52m (5-ft) layer did not appear to be as effective in removing odour as the top of the bed.

Figure 4 - Odour Breakthrough



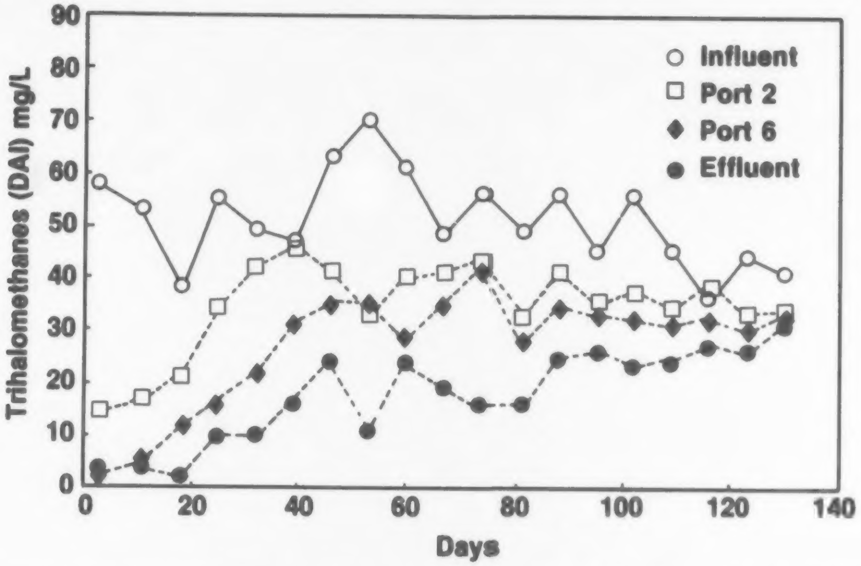
Geosmin was only detectable for 4 weeks in late July and early August in the pre-GAC water at levels of 20-230 ng/L while odours were present throughout and when we did observe an odour breakthrough from the GAC, no geosmin was detected. Similarly, although low-level odours were found in the distribution systems, no geosmin was found. No methylisoborneol was detected in any sample.

Although the algae odours were absent in the finished water we did have a minor odour problem (TON of 2 to 4) due to chlorinous odours formed after post-chlorination of the contactor effluents. The odour formed was a nitrogen trichloride, swimming pool type odour which was evident within two weeks of start-up and appeared to be caused by reaction of the free chlorine with organic nitrogen compounds which were breaking through the carbon. Public response indicated that while people were noticing the chlorinous odour, they found it much less objectionable than the algae odours. Pilot studies showed that aeration for a few minutes with fine bubble air diffusers could eliminate most of this odour, so work is proceeding (1986) on installation of such a system in part of the clearwell.

Trihalomethanes

Trihalomethane levels going on to the carbon were moderate in 1985, peaking at 70 ug/L and dropping to 40 ug/L by November. Breakthrough came early with low levels appearing at the effluent within days of start-up (Figure 5). Levels generally decreased with bed depth, but adjacent segments often showed little difference and occasionally data appeared to indicate some desorption.

Figure 5 - TTHM Breakthrough



Percentage removal through the beds (Figure 6) shows that 0.60m (2 ft) of carbon (port 2) became ineffective very rapidly while the effluent (3.05m, 10 ft) values showed better than 50% removal for almost 100 days although this had dropped to 25% after 130 days. Increases in removal efficiencies at 52,80 and 123 days appear to be related to EBCT (Figure 7).

Figure 6 - TTHM Percent Removal

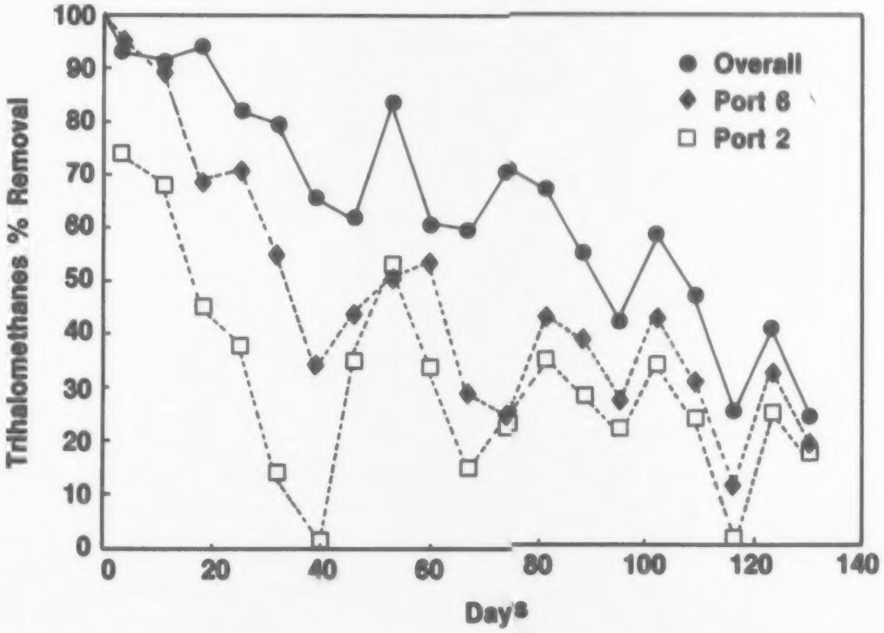
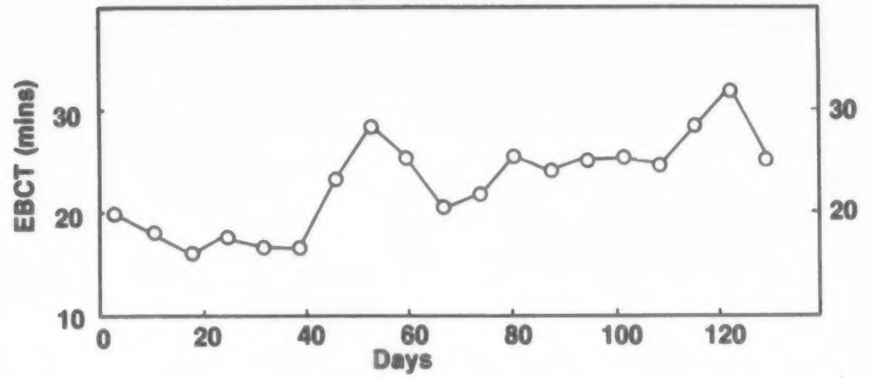
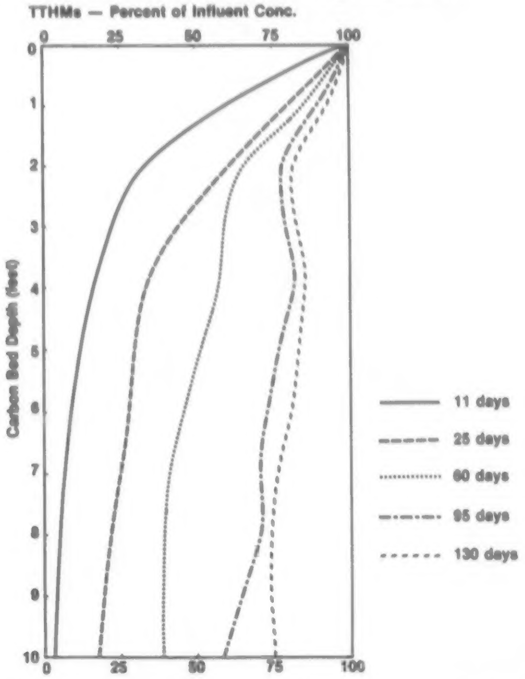


Figure 7 - EBCT



Wavefront progression through the bed (Figure 8) shows that the front is very broad, extending over the whole depth of the contactor with best removals being in the top of the bed, whereas the bottom 1.52m (5 ft) shows only marginal removals from segment to segment. Lines occasionally have a negative slope indicating desorption or TTHM formation.

Figure 8 - TTHM Wavefront Progression



Cumulative bed loadings against days in use (Figure 9) were calculated from weekly values at each segment using weekly flow totals as if the carbon remained in the same segments throughout. Data show that the top segment (0-0.6m, 0-2 ft) has the highest loading (up to 1400 ug TTHM/g of carbon) and that the overall loading for the 3.05 m (10 ft) bed reaches 600 ug/g of carbon. There is some indication of desorption in some of the intermediate segments.

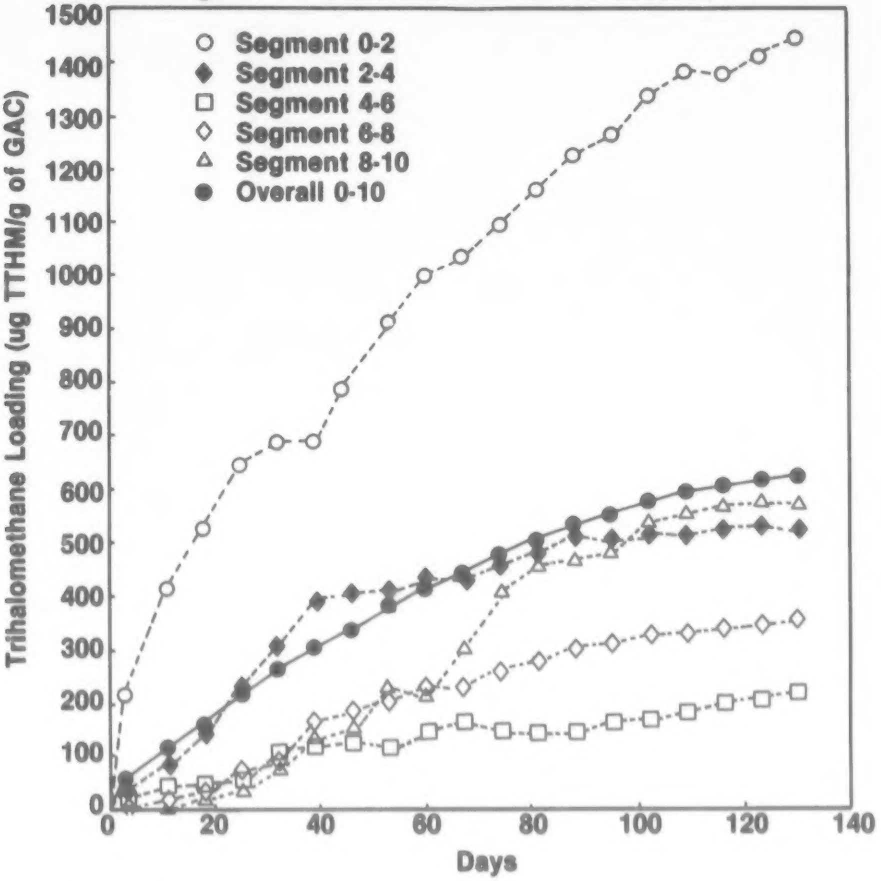


Figure 9 - TTHM Cumulative Bed Loadings

Dissolved Organic Carbon

Levels of Raw TOC were moderate in 1985 at around 6 mg/L C. Conventional treatment reduced this to 2.5-3.5 mg/L going on to the GAC. Breakthrough came early (Figure 10) with detectable levels in the effluent after 10 days. Levels generally decreased with bed depth and effluent levels remained below 1 mg/L C for 60 days and below 2 mg/L for 130 days.

Percentage removals (Figure 11) indicate increases in removal with bed depth with the effluent maintaining 50% removal to 80-90 days and 40% removal at 130 days.

The significant increase in removal efficiency throughout the bed around days 52, 81 and 123 coincides very well with increases in the EBCT to values around 30 mins (Figure 7). Removals thus appear to be very dependent on contact time.

Wavefront progression of DOC through the bed (Figure 12) shows the same broad fronts as in the TTHM case with removal occurring over the whole depth of the contactor, breakthrough very early, and the top of the bed being more effective than the bottom. However, unlike the TTHM situation, the DOC fronts have a more positive slope with no indication of negative slopes (desorption).

Figure 10 - DOC Breakthrough

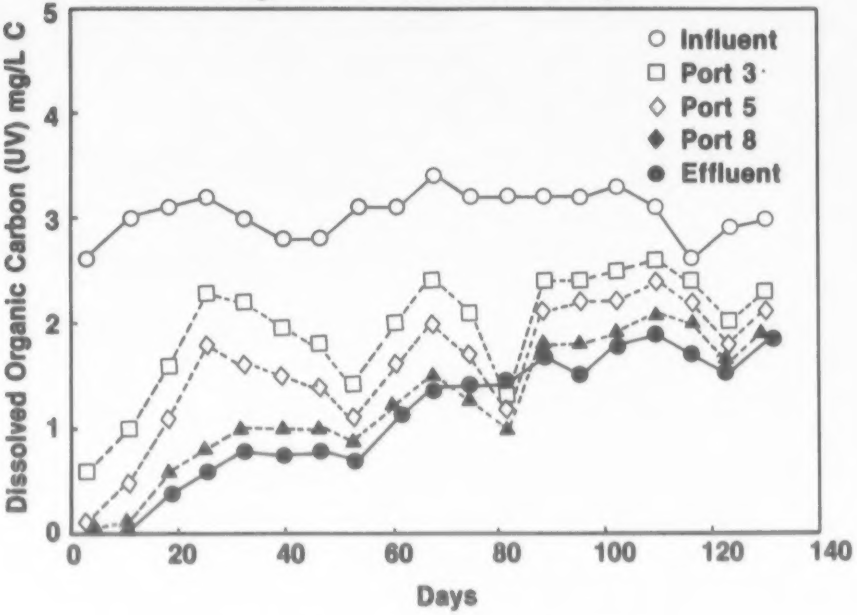
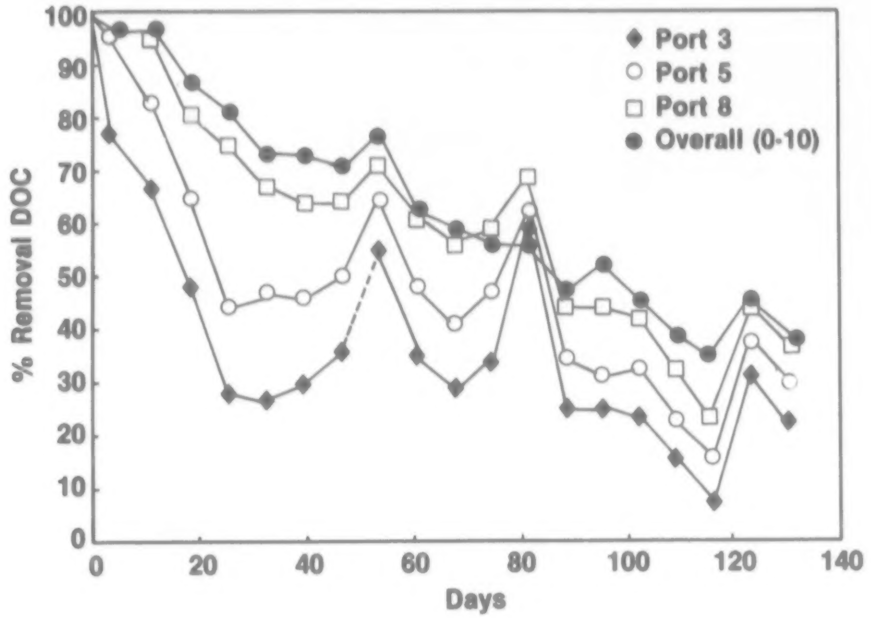


Figure 11 - DOC Percent Removal



Cumulative bed loadings (mg DOC/g of carbon) (Figure 13), calculated as if the carbon remained in the same segments throughout, show that the highest loading is on the top 0.91m (3 ft) segment (6% loading after 130 days) and that loadings decrease with bed depth. The overall bed loading increases gradually to 3.5% at 130 days.

Figure 12 - DOC Wavefront Progression

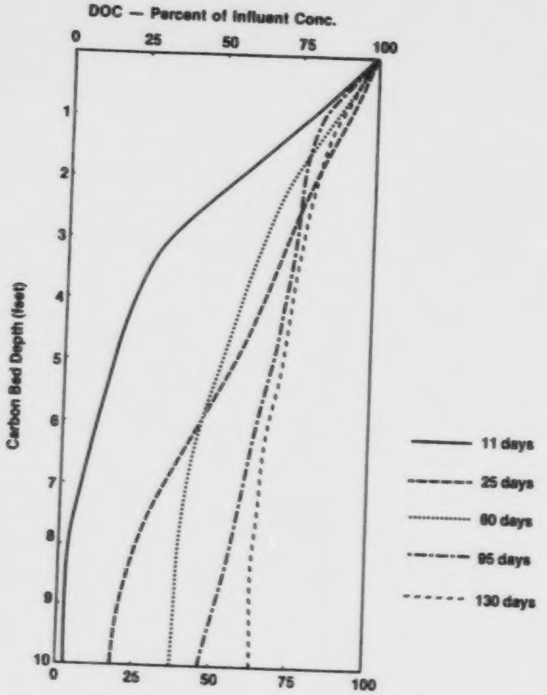
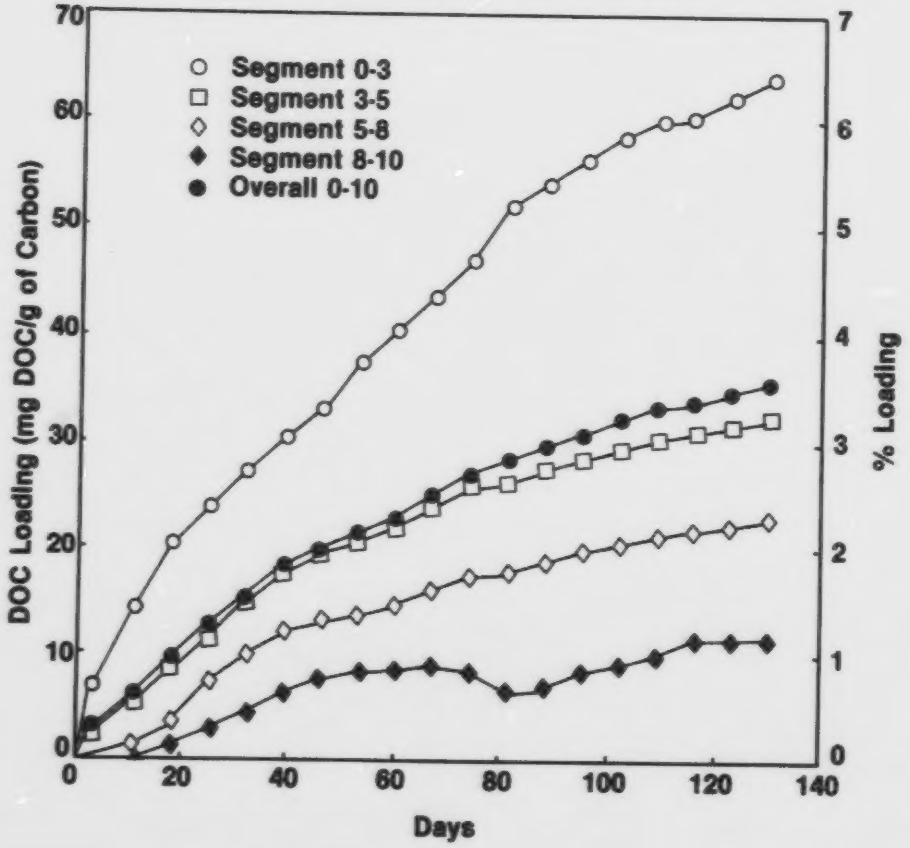
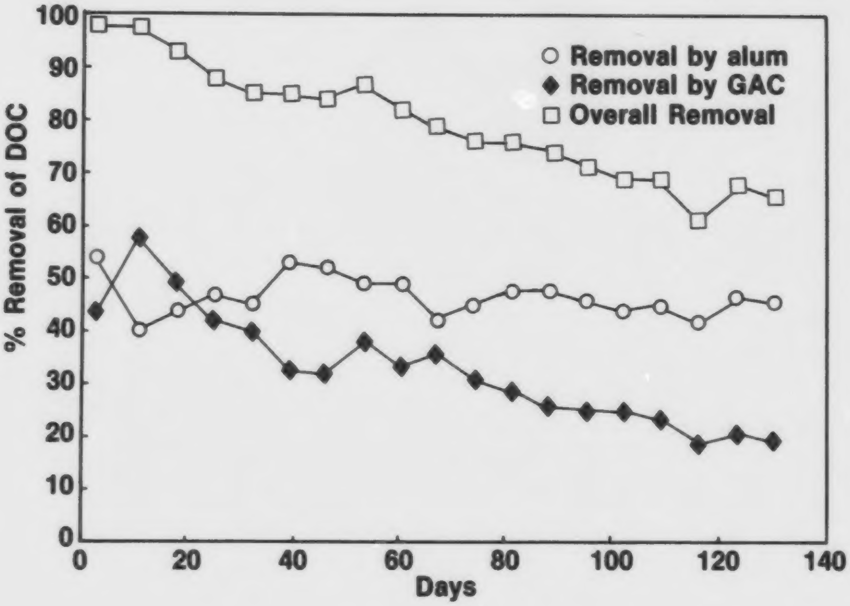


Figure 13 - DOC Cumulative Bed Loadings



In our system, removal of DOC by conventional treatment is as important as removal by GAC (Figure 14). Removal of Raw DOC by alum coagulation/settling/filtration was consistently 40-55% (mean 47%) and while the GAC started off removing the other 45-60% it had dropped off to 20% after 4 months (mean 33%). Overall removal for both processes was good, starting at almost 100% and decreasing to 65%. Optimising conventional treatment will decrease the load on the GAC system and extend its useful life. One important improvement due to the low levels of TOC was the much better residual chlorine in the city distribution systems which led to much lower standard plate counts.

Figure 14 - Percent Removal of Raw DOC



Bacteria

Influent and effluent points for the GAC beds and finished post-chlorinated water were monitored for coliform bacteria and standard plate counts. No coliforms were detected in any of these locations throughout the 130 day period. Standard plate counts for pre-GAC water were generally low (<20/100mls) with a few instances of higher counts. Plate counts in the effluents from the GAC beds were appreciably higher with a geometric mean of 15,600/100mls (range from 50/100 mls to 600,000/100 mls). Contactors containing Calgon carbon had consistently lower SPC levels (mean 2430/100 ml) than the CECA contactors (mean 29,450/100 mls). Post-chlorination reduced these levels generally to < 10/100 mls with a few instances of counts >10 and <100/100 mls.

In 1984, summer monitoring programs on the cities' distribution systems showed 15% of 330 samples had SPC levels exceeding 50,000/100 mls while in 1985 by comparison, only 4% of 378 samples exceeded this level. Geometric means of all SPC samples decreased from 940/100 mls in 1984 to 124/100 mls in 1985.

Comparison of CECA/Calgon Carbons

The two carbons installed in the contactors both worked well and gave very similar results for removal of odour, TOC and TTHMs with odour breakthrough occurring in the same week, TOC effluent levels being +/- 0.1 mg/L and TTHM levels being +/- 2 ug/L in the same week. No noticeable differences were recorded for effluent turbidity, particle counts, pH or for headloss buildup.

The major difference was the presence of phosphate in the effluent from the CECA carbon contactors. Initial phosphate levels of 500 ug/L P were found in the CECA effluents; after 3 weeks levels were still around 100 ug/L p, and it took 9 weeks to drop below 20 ug/L P. During this time, raw water total phosphate levels ranged from 38 - 124 ug/L P. The high levels occurred while using 100% virgin carbon and are not expected to be repeated since the only virgin carbon use in future years will be the 5-15% make-up for losses during regeneration.

With low TOC values and good disinfection in the distribution systems, the high phosphate did not appear to cause us any problems but we would anticipate some concern with areas where biological activity might be stimulated. The higher standard plate counts on the CECA contactors may be due in part at least to the high phosphate levels.

Turbidities/Particle Counts

The GAC contactors had very little effect on the turbidity of the mixed-media filtered water with the mean pre-GAC value of 0.13 NTU dropping to 0.11 NTU for the GAC effluent and finished water means. There was a slight effect in October in cold water operations when the turbidity decrease was significant (e.g. 0.18 to 0.12 NTU), presumably due to removal of particles created by post-flocculation of alum.

Particle counts (2-120 um) were in the range of 200-500/10 mls in the pre-GAC water and this increased slightly to a mean of 870/10 mls in the GAC effluents and had decreased again in the finished water to 480/10 mls. Under microscopic examination, some carbon fines were observed in the GAC effluents but numbers were low and most were < 10 um in size. Filtration of 1-litre samples through 0.45 um membrane filters gave no observable particles or discolouration to the naked eye. GAC contactor backwashes were followed by a 10-15 min. filter-to-waste cycle which appeared to eliminate the majority of carbon fines. Algae cells in the finished water ranged from less than 400/litre in July and August to a peak of 66,000/litre in September and the contactors did not appear to significantly reduce the number of algae which passed the mixed-media filters.

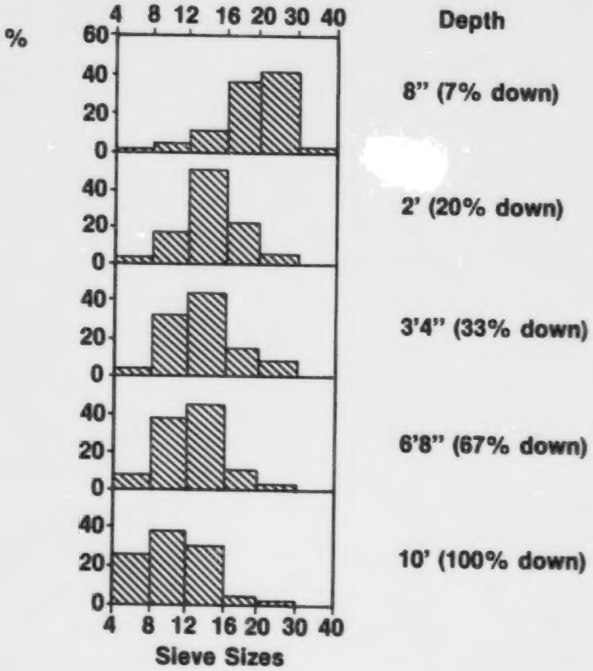
Chlorine Residuals

Chlorine levels going on to the GAC were generally 0.2-0.3 mg/L free and 0.2-0.4 mg/L combined. Free chlorine was never observed (<0.05 mg/L) at port 1 (0.3m) or lower, but traces of combined chlorine up to 0.15 mg/L reached the contactor effluents. Post-chlorination raised the residual levels to 1.0 mg/L free and 0.1-0.4 mg/L combined.

Bed Stratification

Backwashing 3m (10 ft) of carbon produced significant stratification in the contactor beds (Figure 15) with the most noticeable differentiation being in the top 10% of the bed. The top 0.3m (1 ft) is essentially 16 x 30 mesh material (mean size approx. 0.8mm) with the top few inches being very fine. Lower layers are generally closer to 8 x 16 mesh (mean size approx. 1.5 mm). The gradation in mesh size appears to follow the wavefront progression profile in character and may be the reason that removals are better in the top segments of the bed.

Figure 15 - Bed Stratification



Carbon Transfer System

The gravity blowtank transfer system worked reasonably well, especially for the top 2m in each contactor. After that it became progressively more difficult to slurry the carbon into the blowtank and continuous low-rate backwashing had to be resorted to while transferring. The last 0.3m of carbon in between the headers and laterals proved very difficult to remove and required the use of eductors and high pressure water.

Carbon Regeneration System

The Westvaco fluid bed furnace was in operation by January 1985 and fulfilled its 1-day performance specification of 14,000 lbs/day of regenerated carbon at 95% or better of original iodine number and less than 5% smaller than 30 mesh. Regeneration of the carbon took from January 10th to May 5th.

The system is largely computer-controlled and operation was found to be reasonably stable for extended periods of time. Occasionally, some parameter (e.g. bed temperature, plate differential pressure) would cause an upset in the furnace. If this could be corrected in 1 or 2 hours, the system could be brought back on-stream right away but if it took longer it would result in a shutdown which could take 8 hours to 2 days to rectify. Steam quality was found to be critical with any dissolved solids carryover from the steam boiler causing plugging of the furnace gas distribution plates. Variations in carbon feed at the top of the dryer bed and small amounts of sand in the carbon could cause plugging of the dryer gas distribution plate. This often required drilling out the holes in the dryer plate. Sand in the carbon (even at levels of 0.1%) also tended to build up in the regenerator plate section where it interfered with the fluidization of the bed and reduced the maximum throughput. Overall downtime was in the order of 10%.

Our first year of data on regeneration indicates bed-to-bed losses to be 6.5%, somewhat better than anticipated. Carbon could be produced at required specifications with apparent densities and iodine numbers being similar to virgin material (Table 3) although there was a spread in values (+/- 4%). Sieve analyses on the spent and regenerated material showed that the mean size decreased on average by 0.16 mm (range from 0.1 to 0.4 mm) and that the sieve fraction finer than 30 mesh averaged 5.3% (range from less than 2% up to 10%) compared to the virgin specifications call for less than 5%. We will have to wait for the results of this year's operation to see how effective the regenerated carbon is.

Table 3 - Regeneration Data (Means)

Carbon	Iodine Number mg/L	Density Apparent g/ml	Mean Size mm	Effective Size mm
Virgin CECA	1013	0.496	1.45	0.89
Regenerated CECA	991	0.498	1.36	0.76
Virgin Calgon	973	0.530	1.54	0.88
Regenerated Calgon	972	0.529	1.34	0.71

Carbon Usage Rates

From the data collected for Odour, TOC and TTHMs values can be calculated or estimated for the amounts of carbon required to treat a known amount of water to a certain level of exhaustion (Table 4). When the carbon was pulled out of service (135 days) the beds were approximately 70% exhausted and carbon use rates were 50-58 kg per ML (500-575 lbs/MIG). Effective total exhaustion (effluent levels consistently at 95% or more of influent) was estimated at 170-200 days when usage rates would have been in the order of 40-47 kg per ML (400-470 lbs/MIG). Values were also calculated per mg/L of TOC since levels in excess of the 3 mg/L TOC influent concentrations to the GAC would be expected to give proportionately faster usage rates. We hope from this data to be able to roughly estimate exhaustion rates under different water quality conditions.

Table 4 – Carbon Usage Rates

Conditions		Water Treated MIG	Carbon Usage Rate Pounds of Carbon per MIG	Carbon Usage Rate kg of Carbon per ML	Pounds of Carbon per MIG per mg/L TOC**
A. 70% Breakthrough (10 ft)					
Odour	(160 days)	3094*	491	49	164
TTHM	(128 days)	2645	575	57	192
TOC	(140 days)	2866	527	53	176
B. 100% Breakthrough (est.)					
Odour	(200 days)	3654*	416	42	139
TTHM	(170 days)	3234*	470	47	157
TOC	(200 days)	3654*	416	42	139

*numbers asterisked are estimated.
**mg/L TOC refers to filtered water, pre-GAC TOC.
MIG = Million Imperial Gallons
ML = Million Litres

Costs

Capital costs for the whole project are shown in Table 5 and include the GAC contactors, the GAC Regeneration Plant and the Waste Management System. The contactors including pumps, piping, building and carbon cost $7.9 million while the regeneration plant was approximately $3.6 million. The total project cost of $15 million was funded 33% each by the federal, provincial and municipal governments. Capital costs include $1.33 million for the carbon and $1.3 million for the regeneration furnace.

Table 5 – Costs

Capital Costs	Million Dollars (Can)	
1. GAC Contactors (incl. carbon)	7.9	
2. GAC Regeneration Plant	3.6	
3. Waste ponds	3.5	
Total	15.0	
Sub-totals		
1. Engineering Costs	1.74	
2. Carbon (8 contactors plus 1 spare charge)	1.33	
3. Regeneration Furnace	1.30	
Operating Costs (est)		
GAC Contactor Use		
Cost Increase		
— 4 month use - 1.5 cents/1000 gal (imp)	0.3 cents/m^3	
— 12 month use - 2.9 cents/1000 gal (imp)	0.6 cents/m^3	
Regeneration Costs		
4 month use — 3.7 c/1000 gal	(0.8c/m^3)	1 regeneration/year
12 month use — 7.4 c/1000 gal	(1.6c/m^3)	2 regeneration/year

Contactor operating costs which include electric power, natural gas heating, maintenance, salaries and lab costs were 0.32c/m^3 (1.5c/1000 gals) for summer use only (4 months per year) and are expected to be 0.6c/m^3 (2.9c/1000 gals) for year round use.

Regeneration costs (power, steam, natural gas, salaries and maintenance) were calculated to be 0.55 or 1.1c/m^3 (2.5 or 5.0c/1000 gals) for 1 or 2 regenerations per year, not including capital cost recovery or carbon losses. This gives a cost of 24c/kg (11c/lb) of carbon compared to a delivered price of almost $1.76/kg (80c/lb).

Carbon losses were 6.5% for regeneration and transfer which gives an added cost of 11.2c/kg (5.2c/lb) of carbon or 0.26c/m^3 of water treated (1.2c/1000 gals).

Our plant production costs prior to GAC (1984) were 7.5c/m^3 (34c/1000 gals) so the GAC operation (contactors, regeneration and carbon losses) will increase our operating costs by 15%-30%.

SUMMARY/CONCLUSIONS

The new GAC contactor system fulfilled its main purpose in eliminating algae tastes and odours and provided additional benefits of significant reductions in TOC, trihalomethanes and distribution system bacteria levels. It also provided a blanket protection in case of contaminant spills into the water source. Public acceptance of the resulting drinking water was generally very favourable. The limitations of the system have to be appreciated to optimise the operation but the contactors work well with relatively little attention and maintenance. Carbon transferring and regeneration are somewhat more complex and required more time and effort to ensure reasonable levels of operation. Experience and some minor modifications may make these operations more routine.

Some of the limitations are the fast breakthrough of TOC, TTHMs and organic nitrogen with resultant high usage rates for the carbon. The most likely explanation of the broad wavefronts is the disruption of the wavefront caused by mixing of the carbon on backwashing[19,20] with resulting decreases in removal efficiencies. Removals are also affected by particle size and contact time, and unfortunately our worst algae periods usually coincide with our maximum flow rates. It is also difficult to predict parameter breakthroughs and exhaustion since they are not linear with bed depth.

Future Work

One or more years of carbon use will be required before we can tell if the regenerated carbon is as effective as the virgin material and if there are parameters which should be modified in the regeneration process. An 18-month study[21] is underway using measured data from the contactors to computer-model the performance to help make meaningful predictions for use under different water conditions.

In addition we will be doing some exploratory work in the future on the GAC including the effects of backwashing and of particle size, on year-round carbon use, on different carbons, on post-chlorinous odours and on usage rate predictions.

REFERENCES

1. U.T. Hammer, "Blue-Green Algal Blooms in Saskatchewan Lakes", Verh. Internat. Verein. Limnol., 17, pp. 116-125, 1969.

2. W.W. Carmichael (editor), "The Water Environment: Algal Toxins and Health", Environmental Science Research, Vol. 20, 1981. Plenum Press.

3. G. Slater and V.C. Blok, "Isolation and Identification of Odorous Compounds from a Lake Subject to Cyanobacterial Blooms". Wat. Sci. Tech., Vol. 15, pp. 229-240, 1983.

4. Health and Welfare Canada, "Guidelines for Canadian Drinking Water Quality", 1978.

5. Saskatchewan Environment, "Municipal Drinking Water Quality Objectives", 1980.

6. U.S.E.P.A., "Interim Primary Drinking Water Regulations-Control of Organic Chemical Contaminants in Water", Federal Register, Feb. 9, Part II. 1978.

7. L. Gammie, "Use of Granular Activated Carbon as a Filter Medium for Removal of Organics", Western Canada Water and Sewage Conference Bulletin, Dec. 1978.

8. CH2M Hill Canada Ltd., "Water Treatment System Improvement and Expansion Studies", Report to Buffalo Pound Water Administration Board. Dec. 1980.

9. Technical Steering Committee Report (PFRA Engineering Branch) "Regina-Moose Jaw Long Term Water Supply Study - Phase 1", Feb. 1982.

10. G. Muirhead (MLA) Report to Premier "Regina-Moose Jaw Water Supply Study", Nov. 1982.

11. T. Field, G. Giesbrecht (CH2M Hill Canada Ltd), "Taste and Odour Control Facilities for the Regina/Moose Jaw Water Supply", Proceedings of 37th Western Canada Water and Sewage Conference Sep. 1985.

12. R. Van Beek, H.R. Johnson and M.L. Massey, "Start-up and Operation of a Fluidized Bed Regenerator for Granular Carbon used in Industrial Waste Treatment", WPCF 53rd Annual Conference 1980.

13. D.E. Forsyth, L. Gammie and P.M. Huck, "Adsorption of Trihalomethanes in Pilot Scale GAC Columns", Water Pollution Research Journal of Canada, Vol. 17, pp. 23-35, 1982.

14. R.C. Andrews, L. Gammie and P.M. Huck, "Distribution of Volatile Organics in Pilot Scale GAC Columns", presented at 66th Annual Chemical Institute of Canada Conference, Calgary, Alta. June, 1983.

15. APHA-AWWA-WPCF, "Standard Methods for the Examination of Water and Wastewater", 16th Edition 1984.

16. A.A. Nicolson, O. Meresz and B. Lemyk, "Determination of Free and Total Potential Haloforms in Drinking Water", Anal. Chem. 49, 814-819, 1977.

17. B.G. Brownlee, D.S. Painter and R.J. Boone, "Identification of Taste and Odour Compounds from Western Lake Ontario", Water Pollution Research Journal of Canada, 19, 1, p. 111, 1984.

18. R.G. Wetzel, "Limnology", p. 208-211, 1975. W.B. Saunders Co.

19. F. Fiessinger, "Optimization of Organic Removal Through the Overall Treatment Process", presented at the Second National Conference on Drinking Water, Edmonton, Alberta. April 1986.

20. J.C. Crittenden, G. Friedman, D.W. Hand, J. Berrigan and T. Speth, "Design of Fixed Beds to Remove Multi Component Mixtures of Volatile Organic Chemicals", Poster at AWWA Conference, June 1985.

21. P.M. Huck, Research and Development Bulletin, Supply and Services, Canada. Oct. 1985.

USE OF A MOBILE FIELD FACILITY IN THE EVALUATION OF TREATMENT PROCEDURES FOR THE CONTROL OF WATERBORNE GIARDIASIS

by

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I. INTRODUCTION

Giardia duodenalis is an important worldwide intestinal parasite. Infections caused by this protozoan are often asymptomatic but it is the most common intestinal parasite associated with symptoms in Great Britain (Goel et al. 1977) and in the United States (Morbid. Mortal. Wkly. Rpt. 1978). In Alberta, giardiasis is the most frequently reported intestinal infection and notifications are probably a considerable underestimate of prevalence (Alta. Epidemiol. Rpt. 1981). In developing countries, *G. duodenalis* infections have been linked to malabsorption and growth retardation in children.

Transmission occurs either from person to person or via drinking water. The former mechanism is most common in crowded social conditions such as Day Care Centres and Institutions (Black et al. 1977; Keystone et al. 1978). Waterborne transmission has been linked to both endemic and epidemic *G. duodenalis* infections in North America (Meyer and Jarroll 1980). Giardiasis has been associated with contaminated water reservoirs and with recreational camping in Alberta, British Columbia, New York, New Hampshire, Washington and Colorado.

Beavers have been implicated in waterborne infections (Lopez et al. 1980; Dykes et al. 1980; Lippy 1981; Morbid. Mortal. Wkly. Rpt 1977; Wilson et al. 1982) but the role of other wild and domestic animals in the transmission of *Giardia* is unknown. *Giardia* spp. have been detected in a number of animals in southern Alberta including voles, mice, dogs, wood rats, and beaver (Wallis et al. 1984; 1986). Giardiasis is a notifiable disease in Alberta and approximately 125 cases are reported each month. The frequency of these reports suggest the giardiasis is endemic among both animals and humans in Western Canada.

The potential for waterborne transmission of giardiasis to humans through municipal water supplies therefore exists in Alberta and is believed to have occurred at Banff in 1982, Edmonton in 1983 and Exshaw in 1984. A total of 1508 cases of giardiasis were reported in 1985, an increase of 12.5% over 1984 (Alta. Epidemiol. Rept. 1985). Calgary has reported over one third of the notified cases of giardiasis for the second year in a row. For all cases reported, consumption of untreated drinking water was commonly cited as the source of infection.

The Alberta Government recognizes the importance of water for human consumption as part of the basic water management policy embodied in the Water Resources Act. At present, there is very little information available regarding optimum treatment procedures for the removal of Giardia cysts from drinking water under Alberta conditions. Consequently, these experiments and future work will allow guidelines and standards to be set for the province.

The project constitutes major initiatives on the part of the Alberta Environmental Centre and the Kananaskis Centre for Environmental Research (University of Calgary). Its purpose is to test the ability of chlorination, filtration and other treatment procedures to inactivate or remove cysts under portable water treatment operating conditions using a mobile experimental water treatment plant. The work is funded by the Alberta Environmental Centre and the Research Management Division of Alberta Environment.

Work carried out at the Kananaskis Field Station on Giardia culturing, identification and enumeration had reached a point where utilization in applied studies appeared feasible. Concurrently, the Alberta Environmental Centre had designed and constructed a Pilot/Demonstration Plant which provided the physical facilities for undertaking treatability studies. During the fall of 1985, a treatability study was initiated.

II. DESIGN OF AN EXPERIMENTAL MOBILE WATER TREATMENT PLANT

Due to the site specific nature of many water problems and the alterations of water characteristics with sampling, transport and storage, a heavy emphasis has been placed on the development of field or on-site research capability. To encompass various applications, a plant was designed to provide optimum flexibility. The incorporation of alternative technologies into a facility based on conventional treatment (Class 4) provides the ability to subject source waters to a wide variety of treatment options. This versatility provides the greatest potential of arriving at the optimum solution for any given situation. This approach was intended to provide a facility with a continuing useful life thereby spreading the initial investment dollars over a long period of application. The additional advantage of this approach is the capability of rapid configuration of the pilot plant to meet specific requirements at the Alberta Environmental Centre. When ready, it is simply moved to the study site of interest.

The selection of the most efficient pilot plant design depended on many critical factors. These included physical weight and sizing limits for transportation, sufficient output, dependability, versatile treatment for a wide range of water sources, and economic feasibility; both in capital and operational costs. The eventual goal of this pilot plant is to provide research statistics to justify scale-up to a full municipal system that will provide optimum water treatment to match local conditions.

A comparison of initial capital cost showed only a small difference between 136 Lpm (30 gpm) and the 227 Lpm (50 gpm) capacities with the overall weight and dimensions increasing slightly. A 341 Lpm (75 gpm) plant, however, was significantly more expensive and the overall weight of the components nearly doubled. A capacity smaller than 100 Lpm would have required custom built components, making its costs similar to that of a 227 Lpm (50 gpm) plant. Also, this small a plant would have a considerable scale-up factor to full municipal sizing. Consequently, the system was tailored around a nominal capacity of 227 Lpm (50 gpm).

With the selection of the 227 Lpm (50 gpm) plant capacity, the prospective shelter for the components was designed to provide the optimum accommodation for the selected water treatment plant. This plant consists of 2 reactor-clarifiers, 2 filters and a collection reservoir, whose heights normally run about 2.44 m (8') to 3.05 m (10'). The size of the shelter which proved most feasible was 3.66 m (12') wide by 15.85 m (52') long with an overall maximum outer height of 4.75 m (15' 7"). There is a 0.305 m (1') overhang on both sides of the lowboy, but this does not cause any instability since the significant loadings were located along the trailer's centre line.

The initial goal was to select a self-contained trailer with axles and wheels intact. Although the floor area needed for this type of shelter was commonly available, the static weight support and minimum inner height requirement of our treatment plant dictated that our trailer would have to be custom built. Upon notifying several trailer manufacturers concerning our requirements, it was learned that the height could be adjusted but no standard tandem-axle support was available to withstand the 45.41 tonne (50 ton) static load. [Dynamic load = 15.0 tonnes (16.5 tons)].

The obvious solution to the weight problem was to purchase both a lowboy trailer base and a trailer shelter mount. The static weight was primarily supported by the lowboy's frame and an additional three pairs of heavy duty dolly legs. The deck size selected for the lowboy was 3.05 m (10') wide by 15.85 m (52') sitting 1.07 m (42") from deck-top to ground level to conserve height. Since most lowboys of tandem (dual-rear axle) structure are built to be 2.59 m (8.5') wide, the 3.05 m (10') width needed was specially built. This was preferable over building the lowboy to be 3.66 m (12') wide (like the trailer body), since the 3.05 m (10') width is easier to drive, and the rear axles for the 3.66 m (12') trailer would have to be excessively elongated, decreasing their durability and carrying capacity.

To ensure compatibility with municipal plants, specifications were prepared for a range of flow conditions for both clarifiers (Table 1) and filters (Table 2). The final layout of the plant is given in Figure 1.

Table 1

Operating Specifications for each Clarifier

Potable Output		Rise Rate Under Troughs		Retention Time to Overflow
LPM	IGPM	LPM/M^2	GPM/FT^2	MINUTES
45.50	10.0	14.70	0.30	250.0
91.00	20.0	29.40	0.60	125.0
113.75	25.0	36.75	0.75	100.0
136.50	30.0	44.10	0.90	83.3
182.00	40.0	58.80	1.20	62.5
227.50	50.0	73.50	1.56	50.0

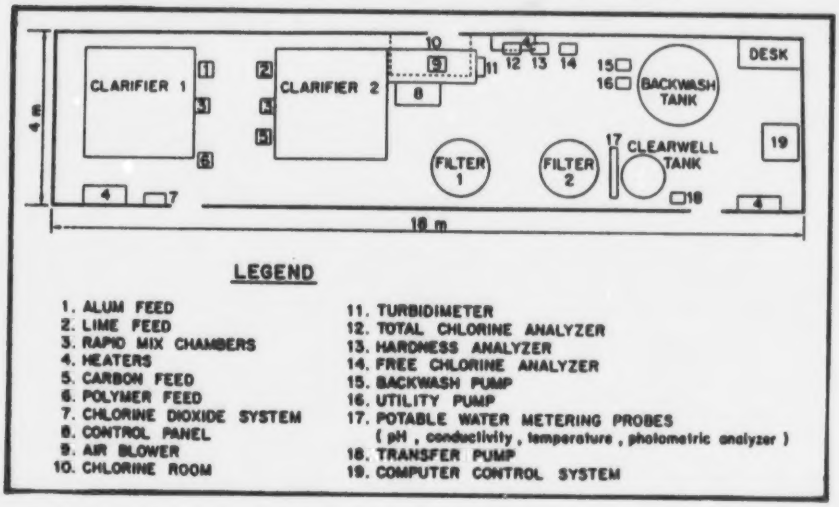
Table 2

Operating Specifications for each Filter

Potable Output		Operation Rate	
LPM	IGPM	LPM/M^2	GPM/FT^2
45.50	10.0	32.81	0.67
91.00	20.0	65.13	1.33
113.75	25.0	97.94	2.00
136.50	30.0	130.75	2.67
182.00	40.0	163.07	3.33
225.50	50.0	195.88	4.00

Figure 1

Layout of Mobile Water Treatment Plant


CLARIFIER 1
CLARIFIER 2
1
2
3
3
5
6
4
7
8
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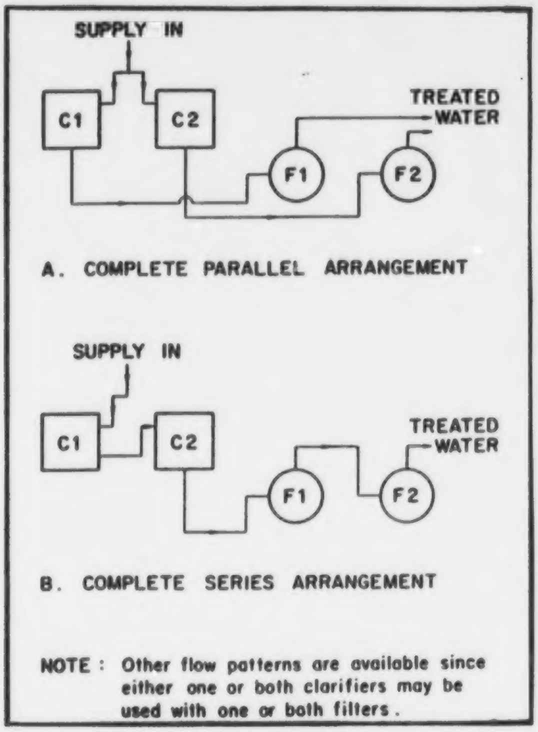
10
11
12
13
14
15
16
BACKWASH TANK
DESK
19
FILTER 1
FILTER 2
17
CLEARWELL TANK
18
4
4 m
16 m
LEGEND
1. ALUM FEED
2. LIME FEED
3. RAPID MIX CHAMBERS
4. HEATERS
5. CARBON FEED
6. POLYMER FEED
7. CHLORINE DIOXIDE SYSTEM
8. CONTROL PANEL
9. AIR BLOWER
10. CHLORINE ROOM
11. TURBIDIMETER
12. TOTAL CHLORINE ANALYZER
13. HARDNESS ANALYZER
14. FREE CHLORINE ANALYZER
15. BACKWASH PUMP
16. UTILITY PUMP
17. POTABLE WATER METERING PROBES
(pH , conductivity , temperature , photometric analyzer)
18. TRANSFER PUMP
19. COMPUTER CONTROL SYSTEM

With the insertion of extensive interconnecting piping and flow control valves, the versatility in water treatment was increased by expanding the number of different sequences in water treatment. The plant contains two reactor clarifiers and two filters, so interconnecting plumbing allows the clarifiers and filters to operate in series, in parallel, or as independent clarifier-filter systems operating side by side at different chemical dosages/flow rates (Figure 2). Clarification times and chemical dosages can therefore be modified to suit different water inlet sources. This highly adaptable unit actually simulates many different water treatment plants but at a fraction of the cost.

Ease of operation was increased with the addition of a microprocessor which remotely controls and monitors the output parameters. This permitted the option of running the plant unmanned for extended periods of time providing that the chemical additives are provided for. A 36 to 48 channel input microprocessor which would monitor and record signals from pumps, thermocouples, turbidimeters, colourimeters, hardness testers, pH meters, flowmeters, chlorine meters, chlorine dioxide, conductivity meters, and chemical feed equipment was therefore built in.

Figure 2

Flow Schematic for Mobile Water Treatment Plant


SUPPLY IN
C1
C2
TREATED WATER
F1
F2
A. COMPLETE PARALLEL ARRANGEMENT
SUPPLY IN
C1
C2
TREATED WATER
F1
F2
B. COMPLETE SERIES ARRANGEMENT

NOTE : Other flow patterns are available since either one or both clarifiers may be used with one or both filters.

III. WATER TREATMENT TRIALS AT THE KANANASKIS FIELD STATION

Introduction

The mobile water treatment plant was moved to the Kananaskis Field Station at Barrier Lake in Kananaskis Country (80 km west of Calgary). The site afforded easy access to an abundant supply of raw water (Barrier Lake) and sewage treatment lagoons for disposal of water contaminated with *Giardia* cysts. When no cysts were present in the plant effluent, water was discharged to the Kananaskis River. All contaminated effluent was superchlorinated at a chlorine concentration of approximately 15 mg/l.

The purpose of the field trials was to investigate the efficiency of conventional water treatment methods in killing and removing cysts. Two series of experiments were carried out. The first was conducted to evaluate the effect of filter maturation on cyst retention by the sand/anthracite filter. The second series of trials was used to determine the effect of chlorine on *Giardia muris* (GM1) cysts.

REMOVAL OF CYSTS BY FILTRATION

Filtration trials were carried out in order to monitor the cyst removal efficiency as the sand/anthracite filter matured.

Methods

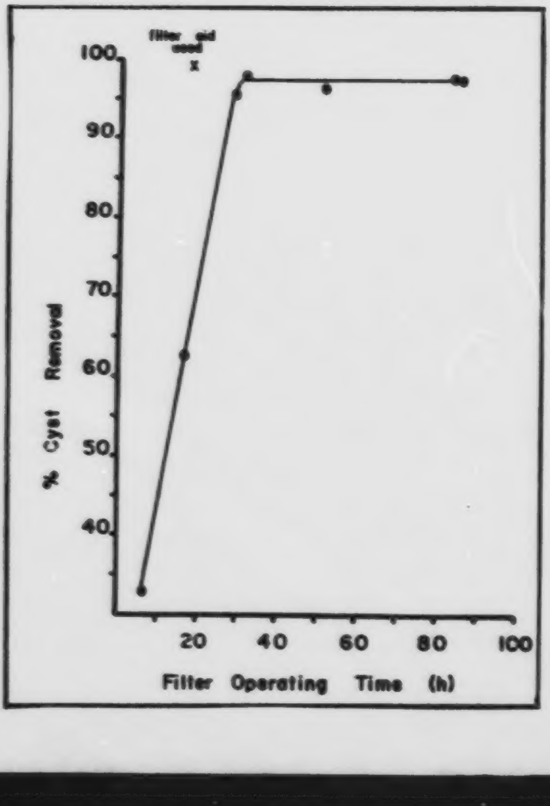
The filter was initially backflushed and then operated at 150 L/min intermittently over a period of 17 days until the filter was too clogged to maintain this flow rate. A total of approximately 760,000 L of Barrier Lake water was filtered. Cyst removal efficiency was tested after 7, 17.5, 29, 31, 51, and 85 hours of operation. At the end of the run, the filter was backflushed and samples were collected and analyzed for cysts. A final trial was conducted when the rejuvenated filter had been operating for 18 hours. This experiment was conducted in the same manner as the preceding trials but polymer filter aid was added to the rapid mix chamber of Clarifier 1.

Strontium tracer and varying numbers of GM1 cysts were simultaneously injected into the water train upstream of the filter in a spike lasting 60 sec. Sequential samples were collected downstream of the filter for 30 minutes in five separate fractions. The strontium concentration was measured in the feed stock, the filtrate, and in samples taken continuously from the post filter effluent. This allowed a prediction of the number of cysts that should have been present in each 6 minute portion of the injected spike if no filtration had taken place. The cysts recovered were counted in each fraction, summed and compared with the predicted numbers [Wallis and Buchanan-Mappin 1984]. All of the recoveries were corrected by dividing the number of cysts by 0.7 to allow for cyst loss during the filtration procedure. When the filter was backflushed, samples were taken of the surface scum and of the backflush water.

Results and Discussion

The results of the filtration trials are reported in Figure 3. Cyst removal by the filter was very low [33%] initially [7 hours], but reached consistently high levels after 30 hours of operation. The maximum removal efficiency observed was 98.6%.

Figure 3 - Filter Removal Efficiency



The experiment with polymer filter aid took place after only 18 hours of operation in a subsequent run and resulted in a removal efficiency of 99%. Cysts were found in the surface scum on top of the filter before backflushing and in the first two samples taken of the backflush water. Eight further samples were negative.

Most of the filtration experiments were carried out without using any form of coagulant. The use of coagulating agents is known to increase cyst removal efficiency so it is encouraging that removal efficiencies as high as 98% could be achieved without it. This series of experiments clearly shows the variability in cyst removal that can be encountered depending on the maturation of the filter. The need for careful disposal of filter backwash water was also demonstrated.

DISINFECTION TRIALS

Disinfection trials were carried out with *Giardia muris* (GM1) cysts and pre-chlorination with gaseous chlorine. Chlorine concentration was continually monitored while the treatment plant was in operation. Chlorine was added to the treatment train between experiments to ensure that the chlorine demand of the clarifiers and piping was kept to a minimum.

Initial studies showed that the concentration of tracer peaked downstream of the first Clarifier 30 minutes after the addition of a spike. Attenuation of the tracer concentration occurred downstream of Clarifier 2 but a plateau concentration was reached after 90 minutes. Cysts were therefore injected over 15 minutes into the rapid mix chamber above Clarifier 1 and recovered by pumping water through four Nuclepore filters at once for an equivalent time at sample sites A and B after 30 and 90 minutes respectively. In this way, four replicate samples were collected for viability testing at sampling points A and B. Viability was assayed using both *in vitro* excystation and animal infection.

Results

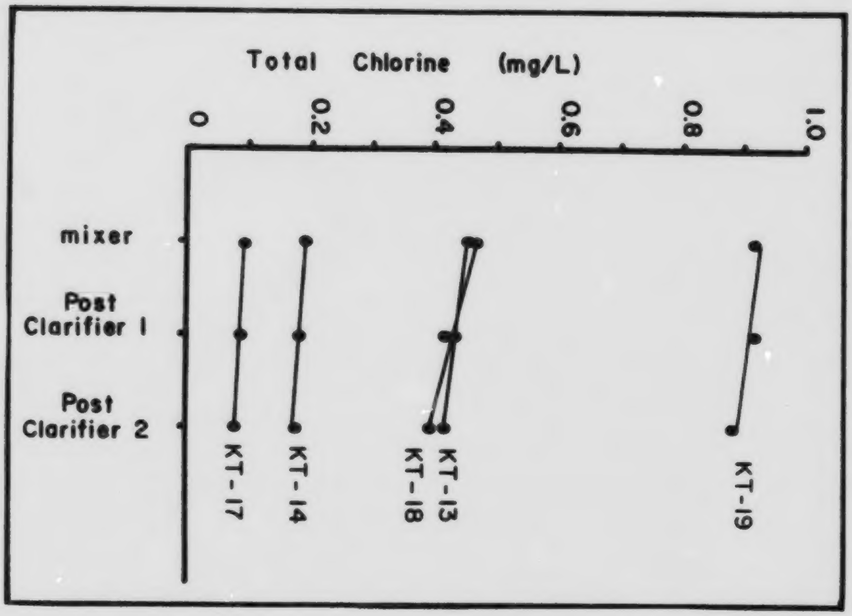
The combined chlorine level of treated Barrier Lake water was found to average 0.05 mg/l and the loss of chlorine in the plant was low (Figure 4). The results of the disinfection experiments are plotted in Figure 5. Very high reductions in GM1 cyst viability were observed at concentrations above 0.15 mg/l with contact times of both 30 and 90 minutes. when the excystation data were plotted against the product of chlorine concentration and contact time (CT), a similar pattern emerged (Figure 6). Mortality rates of 95% or greater were observed at CT values above 13. One trial was carried out using *G. duodenalis* (H8) cysts and slightly higher viability levels were found. At a total chlorine concentration of 0.9 mg/l, 5.3% of H8 cysts were viable after 30 minutes and 1.0% were viable after 90 minutes of contact time. The corresponding CT value was approximately 80, but more experiments may result in a lower value. Animal infection assays confirmed these data.

Discussion and Summary

The results reported above suggest that GM1 cysts (*G. muris*) can be killed in a water treatment plant at chlorine concentrations of approximately 0.5 mg/l and contact times as low as 30 minutes. The results of one additional trail using human *Giardia* cysts suggested that this strain was slightly more resistant. These results are considerably different from those reported in the literature.

Figure 4

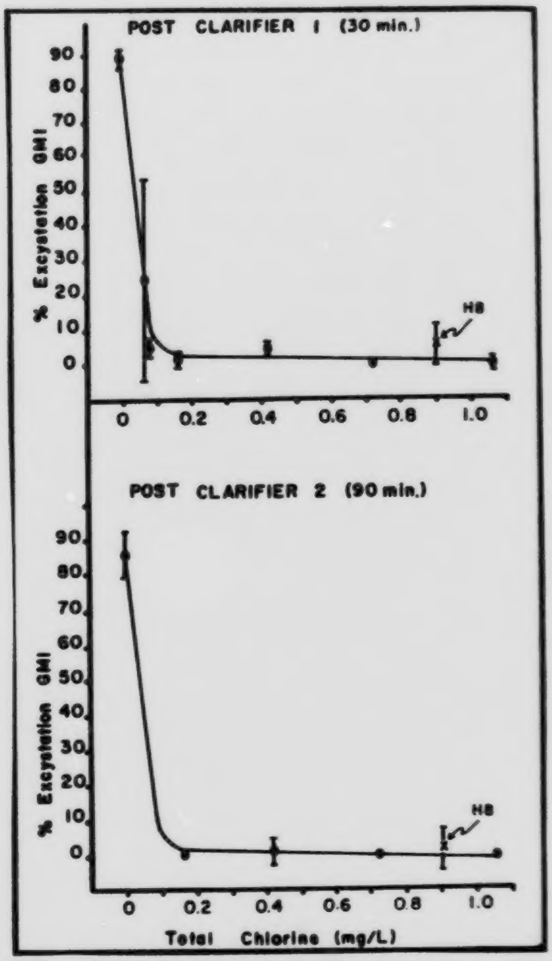
Chlorine Loss in the Mobile Water Treatment Plant during Experiments at Different Chlorine Concentrations



Jarroll et al. (1980) reported that 99.8% of *G. lamblia* cysts could be killed at 3 and 20^{o}C in cloudy river water with free chlorine concentrations of 6 to 8 mg/l. In a later paper, Jarroll et al. (1981) found that 99.9% of *Giardia* cysts could be killed at a concentration of 2 mg/l after 60 minutes (pH 8, 5^{o}C) but that concentrations up to 8 mg/l were required to achieve the same effect after 30 minutes. DeWalle and Jansson (1983) tested the viability of *G. muris* cysts in Banff municipal water (pH 7.8, 1^{o}C) and reported that mortality levels in excess of 99% were possible at concentrations above 2 mg/l and contact times of 90 min. High mortality levels could be achieved at concentrations of less than 1.0 mg/l but only after contact times of 15 h. Rice et al. (1982) found that *G. muris* was slightly more resistant than *G. lamblia* but on the average only about 85% of cysts could be killed after 60 minutes at 2.5 mg/l chlorine (pH 8). Faubert et al. (unpublished data) tested the viability of *G. muris* (the same strain used in this study, GM1) by infecting CD-1 mice with cysts that had been exposed to a total chlorine concentration of 1.18 mg/l and found that animals could still be infected after 60 minutes of contact time. These results showed that at least some of the cysts were still viable.

Figure 5

Viability [% excystation] of GM1 cysts in Barrier Lake
Water [3°C, pH 8.0] as a Function
of Total Chlorine Concentration

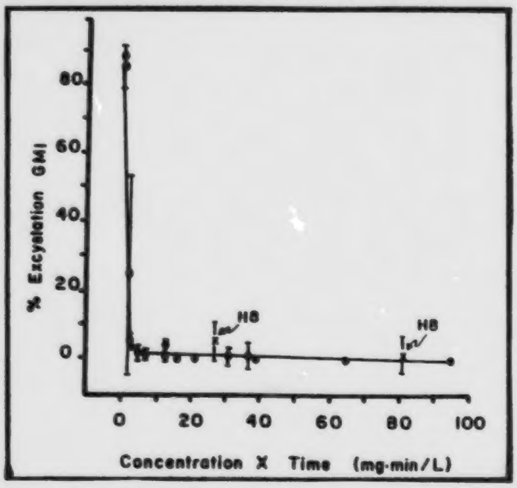


Results reported in the literature indicate that for the conditions existing in Barrier Lake [pH 8, 2-3°C], concentrations of approximately 4 to 5 mg/l would be required to kill 99% of *G. muris* or *G. lamblia [duodenalis]* cysts with 30 to 90 minutes of contact time. The results from our experiments suggest that 99% of *G. muris* cysts can be killed at approximately one tenth that concentration under similar conditions. We believe that the results are valid and the most probable explanations for the discrepancy are either that the effects of pilot plant scale water treatment are significantly different from those obtained on the bench top or that the strain of *G. muris* we used was more susceptible to chlorine than those of previous investigators.

All of the studies summarized above were performed on the benchtop in volumes of water ranging from 200 to 1000 ml. In most cases, the cysts were suspended in phosphate buffer solution with little or no agitation. The cysts were collected with a minimum of disturbance and tested for viability immediately. These conditions could therefore be said to be optimum for cyst survival.

Figure 8

Viability [% excystation] as a Function of the Product of Chlorine Concentration and Time [CT] for GM1 Cysts.



Experiments performed in the mobile water treatment plant were quite different, however. The volumes of water used were 2 to 3 orders of magnitude greater, were continually mixed and circulated, and were recovered by membrane filtration before transportation to the lab for viability testing. The water used was not buffered and contained all of the background contamination typically found in raw feedstock water. It is possible, therefore, that these factors made cysts more susceptible to chlorine. In the control experiment [no chlorine], cyst viability dropped from 88% in the stock solution to 78% after 30 min. and 75% after 90 min. [raw data] suggesting that the treatment plant itself was contributing to the mortality of cysts even in the absence of chlorine. The equipment and procedures used in the mobile water treatment plant were very similar to those used routinely in water treatment plants throughout Alberta.

The second possibility, that the G. muris strain we used was less resistant to chlorine than other Giardia strains is currently being tested in our laboratory. The results of these bench scale tests to date agree closely with other published data. Further testing, both in the laboratory and in the mobile water treatment plant, will be required to resolve these findings.

IV. CONCLUSIONS

1. Methodology was developed permitting the production, quantitative recovery, and viability testing of large numbers of different strains of Giardia cysts for water treatment experiments.

2. Filtration efficiency experiments demonstrated that filter maturation is very important to cyst removal. The sand/anthracite filter was able to remove up to 98% of seeded Giardia muris [GM1] cysts but only after approximately 30 h of operation.

3. Based on a single filtration trial, the use of a polymer filter aid increased the [GM1] cysts removal efficiency to 98% even through the filter had not fully matured.

4. Filter backwash water contained *Giardia* (GM1) cysts indicating that caution should be employed when disposing of such wastewater.

5. Mortality rates of 99% were achieved with the *Giardia muris* (GM1) cysts used in these experiments at chlorine concentrations as low as 0.5 mg/l and contact times of 30 to 90 minutes.

6. Based on a single trial, human *Giardia* cysts responded to chlorine in a manner similar to *Giardia muris* cysts. At a concentration of 0.9 mg/l total chlorine, 94.7 ±6.1% and 99 ±5.3% of HS cysts were killed at contact times of 30 and 90 minutes respectively.

7. The results from these experiments have shown that the mobile water treatment plant is a useful tool for developing water treatment protocols for removing and inactivating *Giardia* cysts at the pilot plant scale. Some of the experiments have yielded results that were not predicted by previous bench scale studies but the investigators believe that the results presented in this report are more applicable to full scale water treatment plants in Alberta.

ACKNOWLEDGEMENTS

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REFERENCES

Alberta Social Services and Community Health Epidemiol. Rpt. 1981. 5:14.

Alberta Social Services and Community Health Epidemiol. Rpt. 1985. 9:477.

Black, R.E., Dykes, A.C. and P.C. Sinclair. 1977. Giardiasis in day care centers: Evidence for person to person transmission. Pediatrics 60:486-491.

DeWalle, F.P. and C.R. Erland Jansson. 1983. Inactivation of *Giardia* by chlorine and UV. Unpublished report to Parks Canada. 24p.

Dykes, A.C., Jaranek, D.D., Lorenz, R.A. Sinclair, S. Jakubowski, W., and R. Davies. 1980. Municipal waterborne giardiasis: an epidemiological investigation. Ann. Int. Med. 92:165-170.

Goel, K.M., Shanks, R.A. McAllister, T.A. et al. 1977. Prevalence of intestinal parasitic infestation, salmonellosis, brucellosis, tuberculosis and hepatitis B among immigrant children in Glasgow. Brit. Med. J. 1:676-679.

Jarroll, E.L., A.K. Bingham, and E.A. Meyer. 1980. *Giardia* cyst destruction: effectiveness of six small-quantity water disinfection methods. Am. J. Trop. Med. Hyg. 29:8-11.

Jarroll, E.L., A.K. Bingham, and E.A. Meyer. 1981. Effect of chlorine on *Giardia lamblia* cyst viability. Appl. Environ. Microbiol. 41:483-487.

Keystone, J.S., Krajden, S., and M.R. Warren. 1978. Person-to-person transmission of *Giardia lamblia* in day-care nurseries. Can. Med. Assoc. J.

119:241-248.

Lippy, E.C. 1981. Waterborne disease: occurrence is on the upswing. J. Am. Wat. Wks. Assoc. 73:57-62.

Lopez, C.E., Dykes, A.C., Juranek, D.D. et al. 1980. Waterborne giardiasis: a communitywide outbreak of disease and a high rate asymptomatic infection. Am. J. Epidemiol. 112:495-507.

Meyer, E.A. and E.I. Jarroll. 1980. Giardiasis. Am. J. Epidemiol. 3:1-12.

Morbid. Mortal. Wkly. Rpt. 1977. Waterborne giardiasis outbreaks. 26:269-275.

Morbid. Mortal. Wkly. Rpt. 1978. Intestinal parasite surveillance - United States. 27:167-168.

Rice, E.W., J.C. Hoff, and F.W. Schaefer III. 1982. Inactivation of *Giardia* cysts by chlorine. Appl. Environ. Microbiol. 43:250-251.

Wallis, P.M. and J.M. Buchanan-Mappin. 1984. Detection of *Giardia* cysts at low concentrations in water using Nuclepore membranes. Wat. Res. 19:331-334.

Wallis, P.M., Buchanan-Mappin, J.M., Faubert, G.M. and M. Belosevic. 1984. Reservoirs of *Giardia* spp. in southwestern Alberta. J. Wildlife Dis. 20:279-283.

Wallis, P.M., Zammuto, R.M. and J.M. Buchanan-Mappin. 1986. Cysts of *Giardia* spp. in mammals and surface waters in southwestern Alberta. J. Wildlife Dis. 22:115-118.

Wilson, H.P.S., Stauffer, S.J. and T.S. Walker. 1982. Waterborne giardiasis outbreak - Alberta. Can. Dis. Wkly. Rpt. 8-20:97-98.

EFFECTS OF MEDIA TYPE ON THE PERFORMANCE OF ANAEROBIC HYBRID REACTORS

by

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INTRODUCTION

The upflow anaerobic sludge blanket (UASB) reactor and anaerobic filters have both been in full-scale operation for the treatment of organic wastewaters for some time (1,2). The UASB reactor consists of a treatment vessel with a bed of dense anaerobic sludge on the bottom and a gas-solids separation unit at the top. Anaerobic filters consist of a bed of support media upon which an anaerobic biofilm attaches itself.

Both types of reactors have problems inherent in their designs. In the case of the UASB reactor, there is a tendency for the washout of biomass under dynamic operating conditions and with various wastewaters (3). The accumulation of excess biomass on the support media in anaerobic filters results in hydraulic short circuiting by the creation of large hydraulic dead zones (4).

As the name suggests, the anaerobic hybrid (anhybrid) reactor is a combination of the UASB reactor and anaerobic filters. First conceptualized by Maxham and Wakamiya (5) and introduced in large scale by DLA (6), the anhybrid reactor is composed of a treatment vessel with an upflow sludge bed zone at the bottom and a zone of support media at the top. Studies have shown that the anhybrid reactor is capable of treating high strength wastes (7) and in some cases attached growth has been achieved on the support media. In all cases the media has been proven effective in reducing the washout of biomass thereby maintaining the biomass in the sludge bed zone.

The anhybrid reactor is currently available commercially and has been utilized in a number of full-scale installations in both Canada and Europe (8). However, there is still a great deal of pertinent information to be obtained in order to optimize the design and operation of the anhybrid reactor (9,10).

SCOPE AND OBJECTIVES

This research was a continuation of the research previously conducted by Oleszkiewicz et al. (9) at the University of Manitoba in which the effect of the depth of the media zone in the anhybrid reactor was examined.

The purpose of this research was to quantitatively determine the effect of type of media on the performance and operation of the anhybrid reactor. It was decided to base the comparison of the types of media on the following criteria:

i) COD removal
ii) SOC removal
iii) cumulative gas production
iv) volatile fatty acids concentration
v) hydraulic characteristics of each reactor

MATERIALS AND METHODS

Reactors

Four parallel laboratory scale anhybrid reactors were constructed from plexiglass tubing. The support media utilized were unglazed one inch ceramic Rashig rings and one inch diameter rigid PVC tubing. The characteristics of the reactors are outlined in Table 1.

Table 1

Physical Characteristics of the Laboratory Scale Reactors

	R1	R2	R3	R4
Height (m)	1.5	1.5	1.5	1.5
Volume (L)	13.4	13.4	13.4	13.4
Media Type	Rashig Rings	Rashig Rings	PVC Tubing	None
Media Zone Volume (L)	5.35	5.35	5.35	-

The reactors were housed in a walk-in environmental chamber set at a temperature of 35°C. The synthetic feed was prepared and stored in an adjoining environmental chamber set at a temperature of 5°C. A thermal shock solids separation device was installed in the second chamber to enhance the solids removal from the effluent and to measure the solids lost from the system. The raw wastewater was pumped to a mixing chamber which combined the influent and recycle flows. The recycle pump then carried the combined flow into the reactor. The effluent flow was separated in a splitter box to remove the biogas before arriving at the effluent container set outside the chamber. Tygon tubing was used in the peristaltic pumps and in all the reactor connections. Figure 1 is a schematic of the laboratory apparatus utilized in the study.

Feed

A synthetic sulphite evaporator condensate (SEC) feed similar to that of Nitchols et al. [11] was prepared in two 100 L containers. The constituents of the feed are listed in Table 2. The influent also contained a trace metal solution which provided 1 mg/L each of nickel, cobalt and iron. Sodium carbonate was added to maintain sufficient alkalinity in the reactors. The influent feed concentration was approximately 5500 mg COD/L.

Analysis

All the tests conducted on the samples obtained during the study were performed according to Standard Methods [12] where possible. Gas analyses were performed on a Gow Mac gas chromatograph. Soluble organic carbon in the influent and effluent was measured on a Dohrmann UV Carbon Analyzer. The biogas produced during the study was measured by low volume gas meters from Triton Electronics. The volatile suspended solids content of each reactor was determined by sampling directly from the reactors through taps that were installed during construction. The solids in the thermal solids shock separator were measured once per week and were not returned to the system. Rhodamine B was used as a tracer in the hydraulic tests and was measured on a Turner Model 110 Fluorometer.

Figure 1

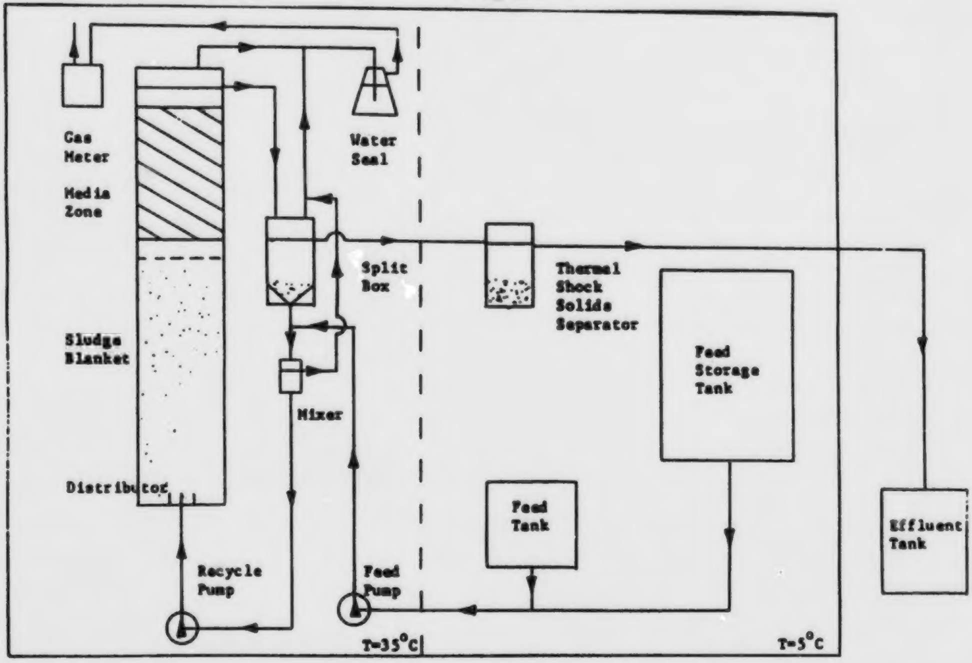

Gas Meter
Media Zone
Sludge Blanket
Distributor
Recycle Pump
Water Seal
Split Box
Mixer
Feed Pump
Thermal Shock Solids Separator
Feed Storage Tank
Feed Tank
Effluent Tank
T=35°C
T=5°C

Table 2

Composition of the Synthetic SEC Used as Feed

Constituent	Concentration (mg/L)
Acetic Acid	2400
Methanol	850
Ethanol	450
Phenol	100
Acetone	100
Ammonium Phosphate Dibasic	10
Ammonium Chloride	10
Potassium Phosphate	10
Magnesium Chloride	10
Furfural	10-25*
Sodium Sulfide	10-25*

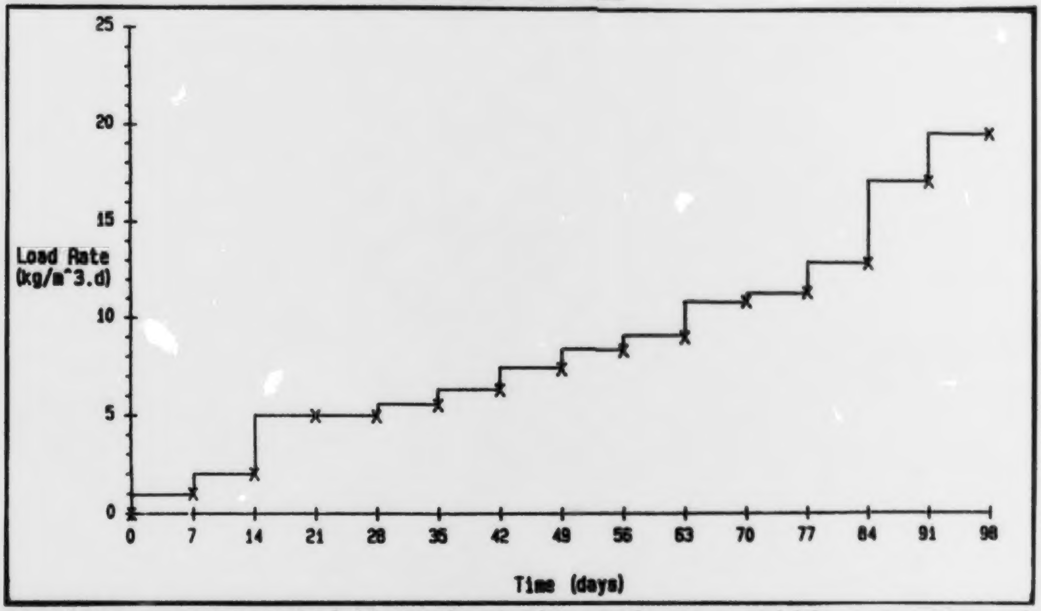
* The concentrations of Furfural and Sodium Sulfide were increased during the course of the study to concentrations that could reasonably be expected in actual SEC. This was done to allow the biomass to acclimate to the compounds.

Operation

The initial sludge for the reactors was obtained from the Winnipeg South End Treatment Plant. Each reactor was charged with 4.14 L of thickened, actively digesting sludge with a volatile solids content of approximately 200 g/L. As can be seen from Figure 2, the initial load was 1 kg COD/ m^3.day at an F/M ratio of approximately 0.1 kg COD/kg VSS.day.

Figure 2

Load Rate vs Time



The reactors were operated throughout the study at a solids retention time (SRT) of approximately 50 days. Generally, steady state conditions are assumed when consistent operation of a reactor is recorded over a period of two or three SRTs. However, due to the obvious time constraints involved in this study, it was assumed that pseudo steady state conditions were achieved after a period of two weeks. That is, the desired removals were achieved for two consecutive measurements and the volumetric COD load was then increased step wise.

The final load at failure of all four reactors was 20 kg COD/m^3.day. The concentration was held constant throughout the study while the flowrate was increased from 2.5 L/d to 55 L/d, the latter at failure.

RESULTS AND DISCUSSION

In order to ensure that all four reactors were operating under identical conditions, the volatile suspended solids content of each reactor and its effluent were monitored. A suitable model was then chosen which characterizes the effects of a variable influent substrate concentration on substrate utilization. It has been suggested by Grau et al. [13] that:

$$dS/dt = K1X(S/So)$$

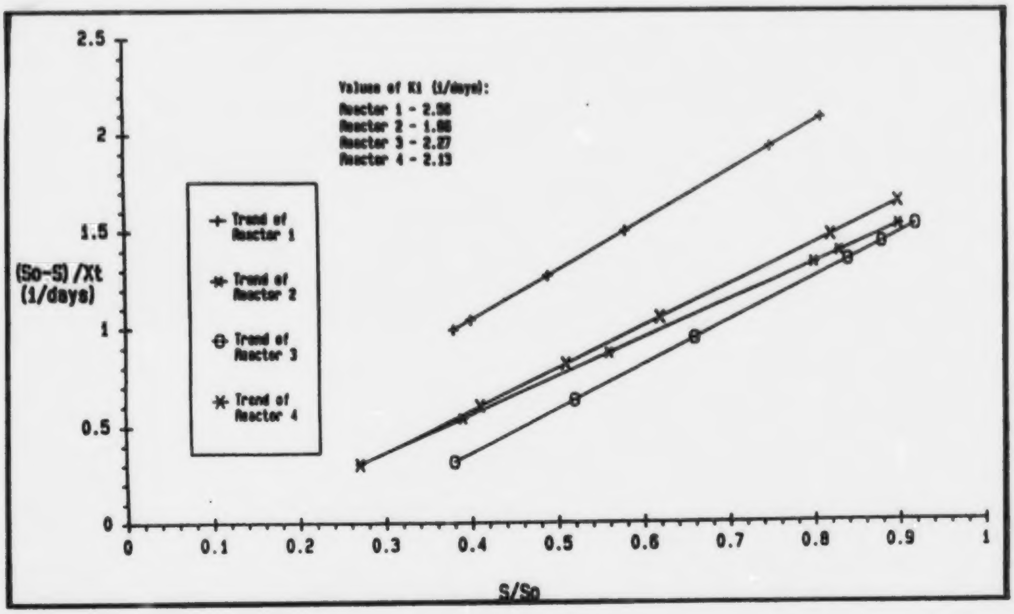
where So = initial substrate concentration (mg/L)
S = substrate concentration surrounding the biomass at any time (mg/L)
K1 = specific substrate utilization rate constant (1/days)
X = active biomass concentration (mg/L)

Thus if a plot is made of (So-S)/Xt versus S/So, the resulting line will give the slope of the specific substrate utilization rate constant, K1. Using this equation, the values or K1 were calculated for each reactor (Figure 3) and are listed in Table 3. It should be noted that the values for all four reactors are comparable, indicating that the reactors were operating under similar conditions.

Figure 3

Determination of Biokinetic Constant, K1

Trends of all Reactors



The COD removal efficiency of the reactors was calculated using the following equation:

$$\text{Removal } (\%) = [(\text{CODi}-\text{CODe})/\text{CODi}] \times 100\% \qquad (2)$$

where CODi = the influent soluble COD concentration
CODe = the effluent soluble COD concentration

Table 3

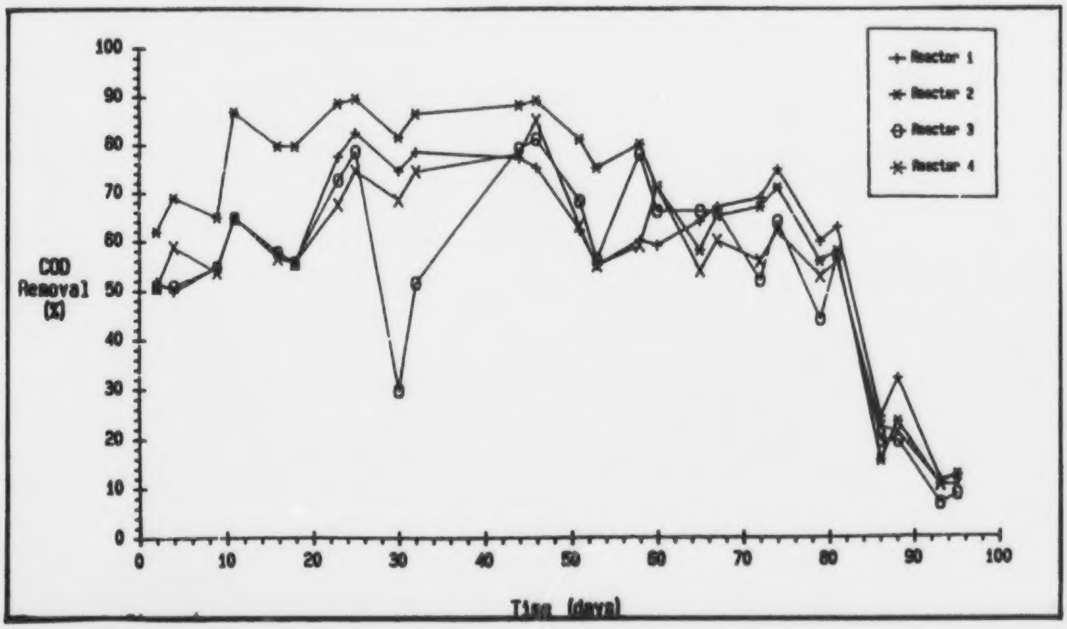
Comparison of Reactor Performance

	R1	R2	R3	R4
K1 (1/days)	2.56	1.96	2.27	2.13
Average COD Removal (%)*	58.9	66.4	54.4	56.2
Average SOC Removal (%)*	58.9	66.0	55.5	52.0
Cumulative Gas Production (L)	3589	3756	3381	2857
CH4 Yield @35%C (L/g COD rem.)	0.37	0.35	0.38	0.31
VFA at Failure (mg/l)	2120	2310	2590	2600

* These values represent the average of the COD and SOC removals recorded for all four reactors over the course of the entire study.

Figure 4

Chemical Oxygen Demand Removal vs Time



As can be seen from Figure 4, the removal efficiency of all the reactors increased as they acclimated to the synthetic feed until day 20 at which point it was decided to increase the load step wise from 5 kg COD/m^3.day until failure was reached in all four reactors. All the reactors maintained consistent removals until day 77 when the load was increased to 13.2 kg COD/m^3.day. At this time a deterioration in the performance of all four reactors was noted with the greatest reductions in efficiency taking place in R3 and R4.

While all four reactors failed at approximately 100 days, it is apparent from Figure 4 that R1 and R2 were providing greater COD removals prior to failure. This is reflected in the average COD removals during the entire study for all four reactors listed in Table 3. The average values for R1 and R2 are both higher than those for R3 and R4. The same holds true for the SOC removals recorded during the study.

It may be observed that, although R1 and R2 possessed identical media and were operating under comparable conditions, R2 had higher average COD and SOC removal efficiencies than R1. This discrepancy can be attributed to an upset that occurred in R1 early in the study due to a mechanical difficulty. Similarly, R3 underwent an upset that required a small period of time for recovery as is illustrated in Figure 4.

Figure 5 illustrates the cumulative production of biogas with respect to time for all four reactors. It is apparent that by day 21 both R1 and R2 were producing greater quantities of biogas than R3 and R4. This trend continued until the end of the study. In all cases the quantity of biogas decreased with a corresponding decrease in COD removal efficiency and this was most noticeable in R4. The values for the cumulative gas production of the reactors are shown in Table 3 as are the values for the methane yield coefficients. R1, R2 and R3 had values greater than 90% of the theoretical value of 0.385 L CH4/g COD removed while that of R4 is 80% of the theoretical value.

The variation in the concentration of volatile fatty acids (VFA) in all four reactors with respect to load rate is shown in Figure 6. The upsets that occurred early in the study in R1 and R3 are visible as is the stabilization that followed the acclimation period. The VFA in all four reactors generally increased with each increase in the load rate. These increases in VFA also correspond to the decreases in COD removal efficiencies seen in Figure 4. By day 80, the VFA in all four reactors had reached levels whereby the normal operating pH of 7-7.5 could no longer be maintained. The pH decreased consistently after that point until day 100. The values for the VFA in all four reactors at failure are listed in Table 3. The values for R3 and R4 are significantly higher than those for R1 and R2 and are an indication of the greater buffering capacity remaining in R1 and R2.

Figure 5

Cumulative Gas Production vs Time

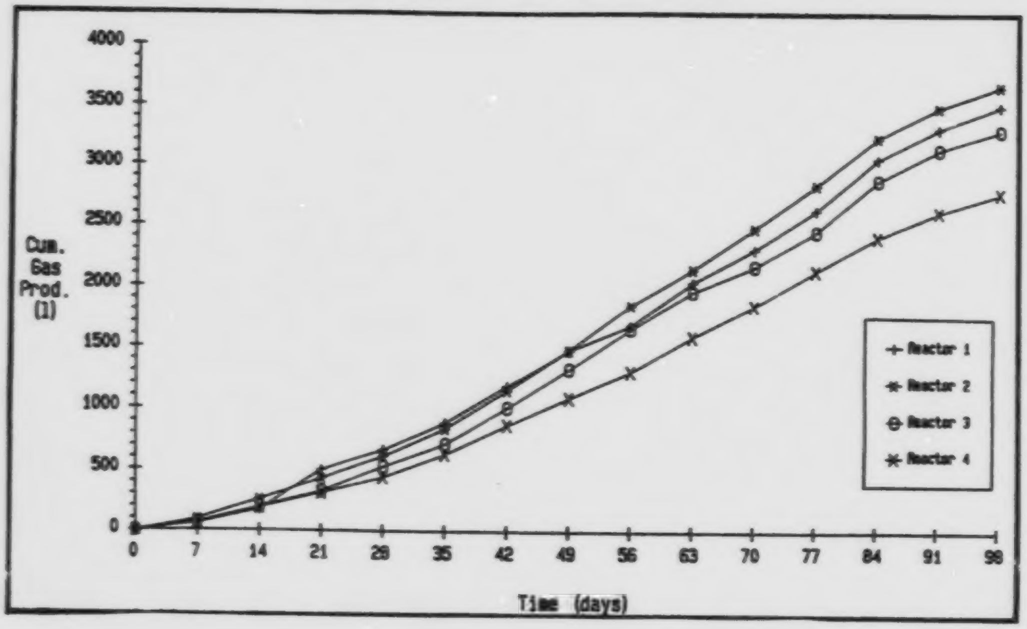
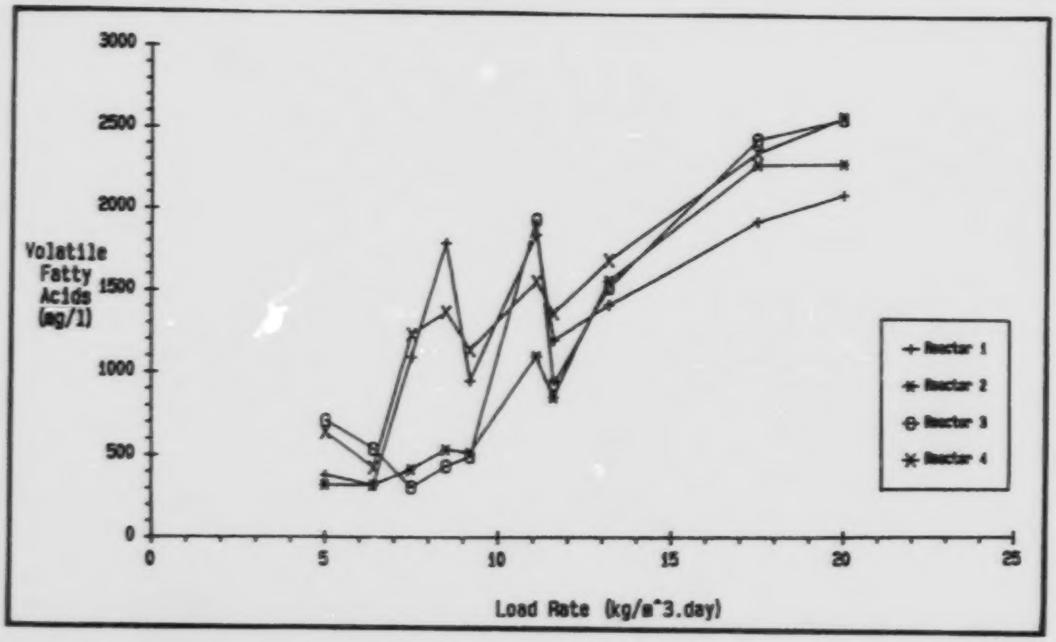


Figure 6

Volatile Fatty Acids vs Load Rate

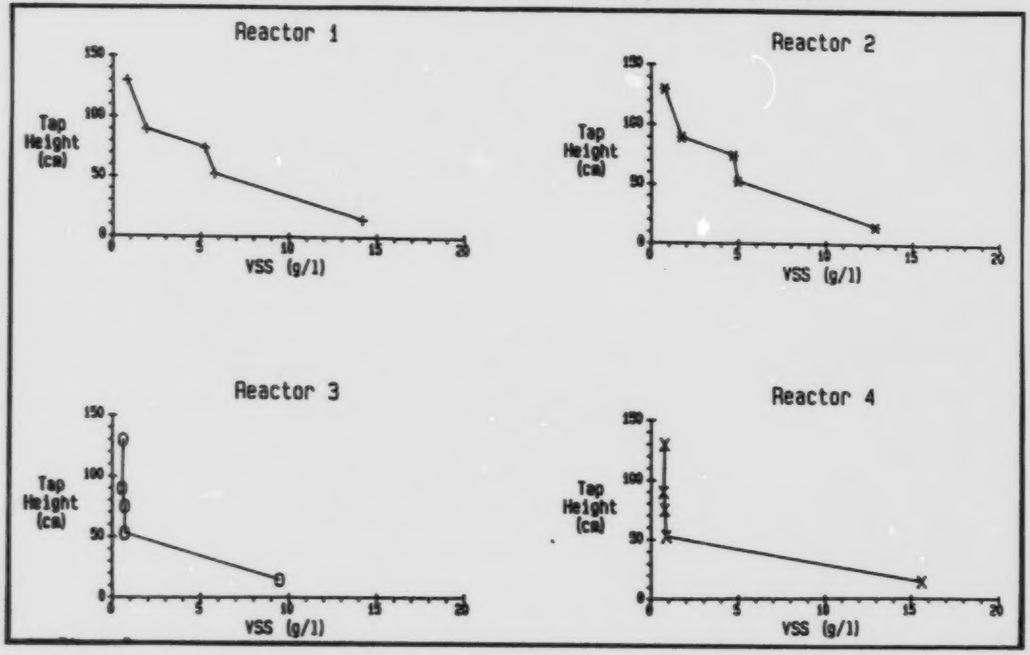


Effects of Hydraulic Parameters on Media Performance

The concentrations of volatile suspended solids in the reactors and their distributions along the height of the reactors were monitored throughout the study. Figure 7 is an example of one of the volatile suspended solids profiles obtained for all four reactors. R1 and R2 retained a greater concentration of fine solids both in the sludge bed zone and in the media volume zone. Thus the significance of the type of media in the anhybrid is clear from the standpoint of start-up and recovery after a shock that may cause a loss in biomass. The random media is effective in retaining biomass and thereby maintaining as low an F/M ratio as possible. The maintenance of a low F/M ratio during the start-up period has been found to be of the utmost importance by Oleszkiewicz et al. (1986).

Figure 7

Reactor Height vs Volatile Suspended Solids



Tracer tests are frequently used in reactor design to help characterize their flow patterns. There are a variety of tracers available including dyes, chemicals and radioactive isotopes. Rhodamine B was chosen as a tracer in this study for its fluorescent properties.

The tracer may be injected into the reactor at a point either continuously or as a slug. In this study the Rhodamine B was injected as a slug directly into the distributor at the bottom of the reactor undergoing testing. The reactor effluent is then monitored at regular intervals to determine the concentration of the tracer in the effluent. From the data obtained, concentration-time curves can be drawn which enable the flow in the reactor to be categorized quantitatively as plug flow, completely mixed or somewhere between the two.

The parameters used to particularize the flow in a reactor include the time to first tracer appearance, the modal time (time at which the highest effluent concentration is recorded], the mean time and the dispersion number. For a test monitored at regular time intervals, t, the concentration, c, of tracer in the effluent is determined. After the desired time has elapsed [usually 4-7 hydraulic retention times], the mean time of the concentration-time curve can be calculated as follows:

$$\bar{t} = \frac{\Sigma tc}{\Sigma c} \qquad (3)$$

The standard deviation of the curve is:

$$\sigma t^2 = \frac{\Sigma t^2 c}{\Sigma c} - \bar{t}^2 \qquad (4)$$

The variance in terms of dimensionless time is:

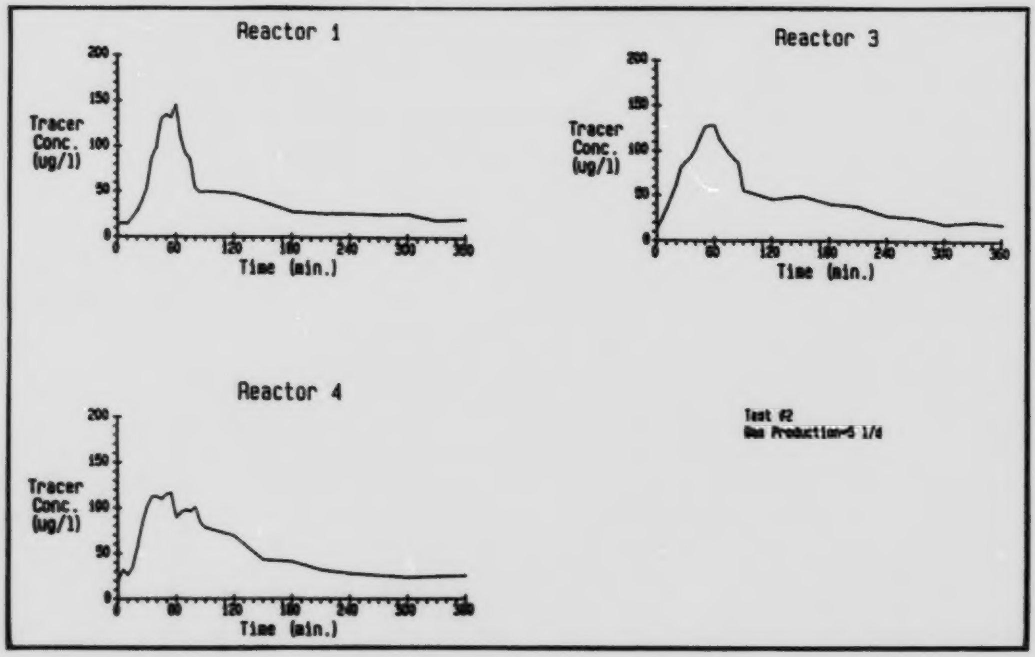
$$\sigma^2 = \frac{\sigma t^2}{\bar{t}^2} \qquad (5)$$

For a closed vessel of finite length, the following relationship has been established:

$$\sigma^2 = 2\left(\frac{D}{UL}\right) - 2\left(\frac{D}{UL}\right)^2 \left(1 - e^{-UL/D}\right) \qquad (6)$$

Figure 8

Tracer Concentration vs Time



The term D/UL is referred to as the dispersion number. The individual terms are as follows:

D = longitudinal dispersion coefficient (m^2/s)
U = mean flow velocity along reactor (m/s)
L = length of path of travel in reactor (m)

For a plug flow reactor the theoretical value of D/UL is zero. For a completely mixed reactor the theoretical value of D/UL is infinity.

The values obtained from the concentration-time curves produced in this study are shown in Table 4. Only R1, R3 and R4 were compared as R2 was identical to R1. The time to the first appearance of the tracer was greater for R1 than for R3 and R4 indicating a greater tendency toward a plug flow regime. Likewise, the modal and mean times for R1 are greater than those for R3 and R4 again exhibiting a tendency towards plug flow. Finally, the variance calculated for each of the reactors is progressively higher from R1 to R4 which in turn indicates that R1 most closely approximates plug flow followed by R3 and then R4.

Table 4

Comparison of Hydraulic Characteristics

	R1	R3	R4
Time to First Appearance (min.)	15	5	5
Modal Time (min.)	60	55	55
Mean Time (min.)	81.0	77.1	78.5
Variance	0.76	0.78	0.85
Dispersion (D/UL)	0.55	0.64	0.84

CONCLUSIONS

The conclusions that can be derived from this study are as follows:

1) the type of media utilized affected reactor performance.
2) the quantity of biomass lost in the effluent of the reactors varied in accordance with the type of media utilized.
3) the reactors with random media (R1 and R2) showed a significantly smaller longitudinal dispersion than the reactor with oriented media (R3) and the sludge bed reactor (R4). Thus the regime in R1 and R2 gravitated towards plug flow while the regime in R3 and R4 showed itself to more closely resemble a random to completely mixed flow regime.
4) the plug flow-like regime was responsible for the effective decrease of biomass build-up time (start-up) in R1 and R2 and this resulted in a higher volume of gas production from the COD removed.
5) the reactors failed after the SEC load was increased to 20 kg COD/m^3.day. Bringing the reactors to failure was intentional and demonstrated the build of higher VFAs in R3 and R4 than in R1 and R2.

References

1. M. Demuynck, E.J. Nyns and W. Palz, Biogas Plants in Europe, D. Reidel Publishing, Dordecht, 1984.

2. A. Wilkie, P.J. Reynolds, N. O'Kelly and E. Colleran, Proc. 2nd Int. Conf. Fixed Film Biological Processes, Arlington, Virginia, Univ. of Pittsburgh, 1984, p. 274.

3. Hall, E.R. Proc. of Min. Envir./Poll. Control Assoc. Ont. Seminar, Bridging the Gap Between Research and Full Scale Operation in Wastewater Treatment, Burlington, Ont., 1984.

4. J.C. Young and P.L. McCarty, Proc. 22nd Ind. Waste Conf., Purdue Univ., 1968, p. 559.

5. J.V. Maxham and W. Wakamiya, Proc. 35th Ind. Waste Conf., Purdue Univ., 1981, p.80.

6. DLA (1981), Anaerobic Treatment of Wastes, Proc. Int. Seminar, Pittsburgh, PA, DLA Inc., p. 300.

7. M. Olthof, W.R. Kelly, G. Wagner and J. Oleszkiewicz, Proc. 39th Ind. Waste Conf., Purdue Univ., Ann Arbor, 1985, p. 697.

8. S. Rekunen, Proc. 1985 Envir. Conf., Canadian Pulp Paper Assoc., Toronto, 1985, p. 25.

9. J.A. Oleszkiewicz, E.R. Hall and Z.J. Oziemblo, Performance of laboratory anaerobic hybrid reactors with varying depths of media, in press, Envir. Tech. Letters, 1986.

10. Ki-Ho Song, J.C. Young, Media design factors for fixed-bed filters, JWPCF, Feb. 1986. p. 115.

11. D.R. Nitchals, M.M. Benjamin, J.F. Ferguson, Combined anaerobic treatment of two waste streams from the sulfite pulping process, JWPCF, March 1985, p. 253.

12. Standard Methods for the Examination of Water and Wastewater, Amer. Public Health Assoc., 16th Edition, 1985.

13. P. Grau, M. Dohanyos, J. Chudoba, Kinetics of Multicomponent Substrate Removal by Activated Sludge, Water Research, Vol. 9, p. 637.

ANAEROBIC TREATMENT OF THERMAL CONDITIONING LIQUOR

by

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INTRODUCTION

The Highland Creek Treatment Plant is located at the mouth of the Highland Creek in the City of Scarborough (West Hill). The plant serves an estimated connected population of 310,000. The original plant had a treatment capacity of 18 x 10^3 m^3/d (4 mgd). Plant extensions to date increased the plant capacity to 216 x 10^3 m^3/d (46 mgd).

The sludge treatment at the plant comprises mainly thermal conditioning of the sludge followed by centrifugal dewatering and multi hearth incineration. Whenever there is an occurrence of high solids in the aeration tanks, some of the waste activated sludge is subjected to thickening by dissolved air flotation and then processed through anaerobic digestion. This is because blending of thickened waste activated sludge with raw sludge prior to heat treatment results in very poor settling after heat treatment. This increases the strength of the decant liquor considerably and causes problems in aeration. From time to time, some raw sludge is also directed to the anaerobic digester to seed the sludge mixture and/or to produce gas to be employed as a fuel in the plant system.

NATURE OF THERMAL CONDITIONING LIQUOR

A mixture of primary, waste activated, thickened and digested sludges is blended and subjected to thermal conditioning using Porteous process. The sludge mixture is held at 190° to 195°C at 1,400 to 1,500 kPa for an average retention time of about 30 minutes. Under these conditions, bound water associated with the solids is released thereby improving the dewaterability of the sludge. The thermally conditioned sludge is allowed to settle in the decant tanks which have a retention time of 3 to 4 hours. The sludge solids are withdrawn from the bottom of the decant tanks, treated with polymer and then dewatered by centrifuging. The decant supernatant and centrifugate are collectively referred to as thermal conditioning liquor. Due to high temperature and pressure involved in the thermal conditioning process, 10 to 35% of the suspended solids are solubilized, thus increasing the BOD content of thermal conditioning liquor to 3,000 - 5,7000 mg/l. Approximately 1100 m^3/d (170 gpm) of thermal conditioning liquor is presently recycled to the head works. The recycle flow comprises less than 1% of the total influent flow (160 x 10^3 m^3/d) to the plant. However, the high BOD content of this stream exerts a tremendous load on the plant representing about an 18% in BOD loading on the system. Since 80% of the BOD loading in the liquor is in soluble form, there is very little BOD removal during primary treatment. This additional load has to be borne by the aeration tanks resulting in increased treatment costs and higher waste activated sludge production. Consequently most of the aeration tanks operate with MLSS concentrations of about 4,000 to 6,000 mg/l. Increased waste activated sludge production and lower primary sludge density in turn increases the number of thermal conditioning units (viz higher costs) and the amount of thermal conditioning liquor recycled back to the plant. Therefore the need to separately treat this side stream for reduction of the BOD and dissolved solids has been identified.

There is another important factor which has to be considered during this project. Highland Creek Treatment Plant has been plagued by odour complaints for some time.

The majority of the complaints are from the residents in the immediate vicinity of the plant. Since all the aeration tanks at this plant are fully covered, the odours occasionally drifting into the neighbourhood seem to emanate mainly from the primary tanks. Increased waste activated sludge production due to thermal conditioning of sludge results in higher return rates and higher suspended solids in the primary tanks, thus overloading primary tanks and causing an odour problem. It is expected that the treatment of thermal conditioning liquor would ultimately cause a reduction of suspended solids in the primary tanks and mitigate the odour problem to a large extent.

BENCH SCALE STUDY

Several researchers have found thermal conditioning liquor to be very suitable for treatment by anaerobic processes[1,2]. Based on their findings, it was decided to treat thermal conditioning liquor from this plant in a modified Suspended Growth Packed Bed Reactor located at Gore & Storrie Ltd. laboratory. The results of this test are given in Table 1 and show an average removal of 88% soluble BOD and a gas production of 0.9 l/g of soluble BOD removed.

Table 1

Average Characteristics of Thermal Conditioning Liquor and Treated Effluent

	T C Liquor	Treated Effluent
Rate l/d	9.8	
S.S. mg/l	980	430
COD_T mg/l	6,600	2,960
COD_S mg/l	6,640	2,270
BOD_{5T} mg/l	4,320	585
BOD_{5S} mg/l	3,098	380
Vol Acids mg/l		195
pH	5.0-6.0	7.9-8.2

Average Gas Production 26.7 l/d

Based on the bench scale test results, Metro Works Department, in conjunction with Proctor & Redfern Consulting Engineers, studied the feasibility of converting two existing anaerobic digesters into anaerobic upflow filters for the purpose of treating thermal conditioning liquor sidestream.

The bio-gas produced by the reactor will be used in hot water boilers for plant heating. Excess gas will be used in the afterburners of the incinerator.

PRELIMINARY DESIGN REVIEW

The filter is designed on the upflow principle with the bottom portion of the filter consisting of a sludge blanket reactor (SBR) and the upper portion a fixed film reactor (FFR) with 6.25 in diameter ProPak biorings as a packing medium. Thus it can be considered that one unit consists of two filters in series. The bottom filter will have the effective sludge blanket occupying approximately 50% of the total SBR height. It was decided that the media will be installed loose rather than in bags. Two filters will be designed; one as 100% stand-by to the operating filter. Each filter is designed to accept maximum anticipated thermal conditioning liquor at 64 mgd plant capacity. A 2:1 recirculation ratio will be provided in the lower portion of the filter. Gas produced will be collected for use in the plant. Overall total BOD removal efficiency will be in excess of 80%.

Design Parameters

@ 64 mgd (BOD_T = 4,300 mg/l COD_T = 8,600 mg/l)
Quantity of thermal conditioning liquor = 1,733 m^3/d
Diameter of the existing digester 22.8 m
Area of digester 408 m^2

Assumptions

Maximum reactor loading	15 kg COD_T/m^3
	7.5 kg BOD_T/m^3
Maximum height of FFR	1.8 m
Rise rate less than	2.0 m/h

Using the above figures, total reactor volume is calculated as 1,713 m^3 giving hydraulic retention time as 1.0 day.

CONVERSION OF THE EXISTING DIGESTERS

There are three primary and three secondary digesters at this plant. However, only two primary and two secondary digesters are currently being used as digesters in the strictest sense. One of the above two primary digesters will be utilised as large holding tanks for the sludge mixture before it is pumped to the blending tanks in the sludge treatment area. It was therefore intended to convert one primary and one secondary digester (both are smaller and older digesters) into anaerobic filters. This would include the following tasks:

-removal of existing equipment in the digester building, including removal of steel floating cover for the secondary digester

-provision of plastic filter media, fixed concrete filter cover, media support

-provision of access hatches on the sides of the filters

-extension of the existing digester control room and provision of three recirculating pumps and two heat exchangers

-construction of new gas compression building complete with three gas compressors and all related gas safety equipment

-construction of one high pressure gas storage cylinder with all related safety equipment

-provision of three new variable speed sewage return pumps, of one new sump pump and of two sludge transfer pumps with related piping

-provision of raw and treated decant liquor piping

-provision of gas piping

The construction of the filter would begin by the Fall of '86.

PRELIMINARY COST ESTIMATES

The major cost of conversion would be associated with the above mentioned tasks. Since the thermal conditioning liquor will be available at over 35°C, no heating (or at the most marginal heating) will be required. All of the gas produced by the reactor would be used in the hot water boilers and afterburners of the incinerators.

Using the above design parameters, approximately 5,000 m^3 of bio-gas (@ STP) would be produced daily. At 80% BOD reduction, 1,490 kg of BOD would be returned to the plant influent. The cost of complete BOD removal must, therefore, include the cost of removing this BOD in the secondary treatment section.

The applicable costs are estimated as follows:

Total cost of conversion including cost of the media	$3.50 million
Annual payment for $3.5 million @ 12%	$0.35 million
Annual O & M cost	$0.40 million
Total annual O & M cost	$0.75 million
Savings on gas produced	$0.15 million
Net annual O & M cost	$0.60 million
Annual cost for conventional treatment of decant liquor	$1.80 million
Annual savings from anaerobic filters	$1.20 million

Thus, with a total investment of $3.5 million and an annual saving of $1.2 million, it would take less than three years to pay off the conversion cost. In addition, the social cost savings through reduced odour problem cannot be adequately quantified and would offset costs involved in this project to a large extent.

SUMMARY

Conversion of existing digesters into anaerobic filters is a viable and cost-effective method of treating high strength thermal conditioning liquor and would significantly improve the plant performance at Highland Creek Treatment Plant.

REFERENCES

1. Crawford, G.V., T. Alkema, M. Yue and M. Thorne, "Anaerobic Treatment of Thermal Conditioning Liquors", Journal Water Pollution Control Federation 54 11, pp. 1458-1464.

2. Hall, E.R. and M. Jovanovic, "Anaerobic Treatment of Thermal Sludge Conditioning Liquor", Proc. 37th Purdue Industrial Waste Conference, West Lafayette, Indiana, May 11-13, 1982.

UNACCOUNTED-FOR WATER - A COMPREHENSIVE APPROACH TO WATER AND REVENUE RECOVERY

presented by

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In determining the efficiency of a culinary water system, auditing the production vs. sales of water, or metered ratio, assists in determining the unaccounted-for in our water systems.

Community Consultants, in conjunction with Brigham Young University, was awarded the Research Grant by the AWWA Research Foundation to perform the research and prepare the recommended procedure for the reduction of unaccounted-for water and leak detection methodologies. This research will be reviewed as part of this presentation.

Customer metering practices for the Water Utilities, by and large, has neglected a substantial resource for additional income by not developing within their utility a thorough meter maintenance and meter operational surveillance program. For the past 10 years, Community Consultants has been doing research on the accuracy of various sizes and customer group types of meters in the utilities throughout the United States.

SCOPE-REDUCTION OF THE ACCOUNTED-FOR WATER - WATER AUDIT

The purpose of this paper is to assist water utilities, both public and private, in understanding the methodology used for determining the efficiency of the culinary water system. It is the intent of this paper to help water utilities, both small and large, to determine the unaccounted-for water potential, sometimes referred to as the metered ratio. The procedure is typically called the Water Audit. The audit uses the same technology as a financial audit except that units of water are audited rather than dollars. The following four basic categories will be addressed:

1. Accountable water.
2. Water loss through inaccurate meters.
3. Water loss due to leaks in the water system.
4. Water loss through miscellaneous uses.
5. Water loss through inaccurate billing and accounting.

The "how to's" will be discussed for determining the efficiency of an individual system and how to correct the problems creating the unaccounted-for water and associated revenue loss.

GOALS

It should be the goal of every water utility to account for 85 percent of the water produced. In accounting for this water, significant increases in revenue will occur, further supporting the budgeting process of providing funds for the following:

1. Operation and maintenance
2. Rehabilitation costs of the existing water system
3. New capital improvements required
4. Retirement of debt
5. Contribution to general fund in lieu of franchise tax

Every water utility should be financially self-supporting and distribute the cost of the operation equitably and fairly to all users on the system.

CULINARY WATER LOSS REDUCTION: THE UTILITY'S APPROACH TO CONSERVATION

Of all the water in the world, only 3/10 of 1 percent is available to man for human consumption. When we consider the water stored in the Polar Ice Caps and in the massive oceans, we are very limited in the amount of water used for the sustainment of life and which can be considered property for man. It is the water for human consumption that is the main ingredient needed for the sustaining of the life of man, the growth of mankind and even the improvement of the values of our natural resources other than water, particularly those of land. The value of land itself is determined by the availability of water to develop that land to its highest and best use. Therefore, as we see the growth of our cities and towns, the major challenge we have is to provide adequate water for the sustaining of the life of man, both through the need for human consumption as well as for fire protection.

The critical needs of our water utilities are seldom recognized. In order to partially understand the significance of our water utility as it relates to our wastewater systems, we can consider the effect of a water outage, such as is experienced with power outages, for several hours or days. Significant problems would occur both for the need of water as a sanitary function as well as in the protection of our lives and properties from fire. We could survive on alternate liquids for a short period of time. However, our sanitary facilities and fire protection would be extremely hampered with the losses of our culinary water systems.

Our research shows that in studying several hundred water utilities, primarily in the West, but throughout the United States, that the average water utility can only account for 58 percent of the water which they produce for culinary uses. Therefore, 42 percent of the water produced is unaccounted for. This research indicates the following:

1. Approximately 18 percent of the unaccounted-for water is lost in leaks in the water system and only 15 percent of those leaks ever surface.

2. Fourteen percent of the unaccounted-for water is lost through inaccurate meters. A significant amount of water goes through meters without being registered.

3. An additional 10 percent of the unaccounted-for water is lost through miscellaneous causes such as malfunctioning equipment, unmetered connections, and poor billing and accounting procedures.

WHAT SHOULD BE YOUR GOAL?

The American Water Works Association Committee on Leak Detection has determined a standard for us to follow. The standard is that utilities using 100 to 125 gallons or water per person per day should account for 95 percent of the water which is accurately metered into the system. If the utility is a highly industrialized utility, then 100 percent of the water sold to customers should be accounted for.

Through research, it is found that the average water use per capita is approximately 100 gallons per person per day. A typical community occupancy is 3.5 people per connection. Therefore, the average daily use per connection would be approximately 350 gallons per day. These averages are for the general area of the United States. Areas in the United States demanding more outside irrigation, such as arid areas or where the amount of turf area per living unit is high, will have an effect on the average daily use. In the United Kingdom, the average daily use per capita is 40 gallons per day. Research done here in the United States in a controlled area determined that the average daily use within the residence was 87 gallons per day per person measuring both water coming into the controlled area and sewage going out of the controlled area in a non-irrigation season.

An example of average daily uses in arid areas is exemplified by the water use comparisons in the cities of Tucson and Phoenix, Arizona. The current average daily use per capita in Tucson is 147 gallons per person per day. In Phoenix, use per capita is 267 gallons per person per day.

In the general area of Colorado, from studies particularly on the West Slopes, we estimate the average daily anticipated usage per capita (based on a one year usage rate) should be in the range of 125 to 175 gallons per person per day for a normal residential community having typical outside irrigation.

HOW DO WE PERFORM A WATER AUDIT?

A water audit is not a difficult procedure, if broken into the following steps:

1. Perform a preliminary water audit.
2. Perform a testing program on the source of supply meters.
3. Perform a testing program on the commercial and industrial meters.
4. Perform a sample testing program on the residential meters.
5. Perform a leak survey.
    a. Leakage potential survey
    b. Gate valve operation survey
    c. An efficient mapping program of the water system
6. Perform a miscellaneous loss survey to determine the efficient operation of the water utility.
    a. Meter reading procedures
    b. Billing and accounting procedures

7. Perform a water rate study to determine the budgeted needs.
    a. Operation and maintenance
    b. System rehabilitation
    c. Capital improvement
    d. Equitable revenue assessments

These seven areas will be discussed in detail on the following pages.

THE PRELIMINARY WATER AUDIT

In performing a preliminary water audit, the total production must be taken for a period of time, (such as a year, month or day) and be divided by the total population served by the utility. This must be divided by the number of days served. The answer will give you the total average use per capita per day. An example of this is illustrated:

$$\frac{\text{TOTAL PRODUCTION OF WATER}}{\text{TOTAL POPULATION X NUMBER OF DAYS}} = \frac{\text{AVERAGE PER CAPITA USE}}{\text{PER DAY}}$$

Hypothetical Example

$$\frac{\text{1,000,000 GALLONS OF WATER}}{\text{10,000 POPULATION X 1 DAY}} = \text{100 GALLONS PER PERSON PER DAY}$$

To refine this further, analyze the utility by seasons. For example, take the amount of water produced from the sources of supply on a winter day. In a normal residential community, this should not exceed 100 gallons per person.

Do the same for a summer day (when there is significant outside irrigation). Take the total production of water, divide it by the total population served and the total average summer daily use per capita is given.

This is the methodology to determine average per capita daily usage based on production versus population. This method allows for water usages to be averaged out for commercial and industrial services.

A more refined approach would be to take the total water production for a period of time and divide it by the total water sales (in units of water) for the same period of time. This basically establishes the metered ratio or the ratio of water produced to that which is sold. This ratio should 95 percent. The method of determining this is as follows:

$$\frac{\text{WATER PRODUCTION}}{\text{SALES}} \text{ X } 100 = \text{PERCENT OF METERED RATIO}$$

It is essential to test the master source meters to make sure the production records are accurate. This, most probably, will have to be done by a specialized consultant with the necessary equipment to test the meters in place to determine their accuracies, repair them, and recalibrate them to a standard accuracy range.

UNACCOUNTED-FOR WATER PROBABILITY

To estimate the total daily leakage on unaccounted-for potentials, the following equation is applied:

```
     DAILY USE PER CAPITA
   - AVERAGE OF 150 GALLONS/PERSON
     -----------------------------
     TOTAL/CAPITA LEAKAGE POTENTIAL
   X NUMBER OF PEOPLE SERVED
    ------------------------------
     UNACCOUNTED-FOR POTENTIAL LEAKAGE
```

Hypothetical Example

```
        290
      - 150
        ----
        140 gallons/person/day
     10,000 people
     ------
  1,400,000 gallons of unaccounted-for water probability per day
```

It should be pointed out that not all unaccounted-for water can be recovered. However, all but 5 percent of the total production can usually be recovered.

If the production meters are accurate and the usage rate is less than 150 gallons per person per day, no further investigation is needed. However, if there is a disparity found between the average daily per capita production and 150 gallons per day, then other steps should be made for corrections.

COMMERCIAL AND INDUSTRIAL METER TESTING REPAIR AND RECALIBRATION

If the preliminary water audit indicates that the unaccounted-for water is high, the next logical step is to proceed with repair and recalibration tests of all commercial and industrial meters (3-inch and larger).

A typical community will sell 50 percent of their water to 10 percent of their large metered customers. Therefore, it is essential to insure that these meters are functioning within a standard accuracy range. Through testing of several hundred meters, it was determined that only 28 percent of the 3-inch and larger commercial industrial meters were operating within the acceptable accuracy range of 95 percent to 102 percent. Significant revenues were lost through the large meters. Therefore, it is extremely cost effective to begin a test and repair program for the large meters.

The revenue typically produced by an effective test and repair program for commercial and industrial meters will produce enough additional revenue to pay for the total water audit program. A utility generally realizes at least $3.00 of recovered revenue for every $1.00 spent in increasing system efficiency by performing a water audit.

Large meter maintenance is essential to your water utility's financial needs. Testing and repairing large meters can significantly increase water sales revenue and will pay for itself over a period of a few months. This is especially true when customers have high water usage.

Although a meter repair program ensures the meter will run in the immediate future, each meter should be continuously monitored for accuracy and total consumption. This is an innovative approach to meter maintenance and has proven to be very cost effective in many cities.

Community Consultants has developed a four-point plan that will provide an effective long-term program for your meter maintenance to ensure high revenue returns.

A FOUR-POINT PLAN

1. MAKE A METER SURVEY AND INSPECTION OF EACH METER

    A. Is there a test plug?
    B. Is there a bypass?
    C. Are there isolating valves on both sides of the meter?
    D. Is the meter installed properly?
    E. Is the correct type of meter installed?
    F. Retrofit meters that are inadequately installed.

2. TEST THE METER FOR ACCURACY

    A. Begin the test from 0 GPM and test through the meter's capacity.
    B. Repair if necessary.
    C. Change the meter or retrofit if not repairable.

3. RECORD THE USAGE RATE FOR 24 HOURS

    A. Compare the historical usage rate with the test accuracy results.
    B. Perform a 24-hour (or more) continuous recording of meters that are suspected of being missized or of malfunctioning.
    C. Review the usage rate at the change-over point with the accuracy of the meter at that flow rate.
    D. Change the meter size or retrofit if necessary.

4. PREPARE A LONG-TERM SURVEILLANCE PROGRAM

    A. Manual surveillance.
    B. Computer surveillance.
    C. When the meter performance is not consistent, the meter will be inspected for:

        1. A change in customer usage rates.
        2. Meter malfunction.

RANDOM SAMPLE IN-PLACE TESTING OF RESIDENTIAL METERS

A utility can very easily perform an in-place testing program for determining the accuracy of residential meters. The recommended flow rates to test are as follows:

Flow Rate	Quantity
1/2 Gallons per Minute	10 Gallons
5 Gallons per Minute	20 Gallons
Full Capacity	50 Gallons

A standard 5/8 x 3/4 inch meter that has been tested and calibrated [to insure its accuracy] with a short length of hose attached can be connected to an outside hose bib of the residence or business whose meter is to be tested. It should be requested that the occupant not use any water for a few minutes while the meter is being tested. There are benefits to this test; one being a determination of leakage between the meter and the residence, and of course, the determination of the accuracy of the meter at low flow rates. It is essential to start the test at the low flow rate rather than the high flow rate. This will ensure a random sample of the accuracies of the meters at their normal operating condition. Beginning with an initial high flow rate would flush the meter out and would not give a true sample of the current condition of the operation and efficiency of the meter being tested.

LEAK SURVEY

A leak survey probably would require the assistance of a consultant to perform the more technical functions of the survey. This includes pipeline location, valve operation and leak locations. It is essential to determine what the leakage potential is by determining the nighttime flows or the flows occurring between midnight and 3:00 a.m. for the total utility as well as for specific zones within the utility that are controlled through an access point by operating the system gate valves. Nighttime flow rates are essential in determining leakage potential in various zones. The method of zone measurement will greatly reduce the cost of locating leaks.

In performing a leak survey, it is essential to initiate a valve operation program. This enables the valves to be operated to isolate zones for performing flow measurements in determining leakage potential. The side benefit of a valve program is that all valves in the system are open and functioning properly. Case studies have shown that 25 to 35 percent of all valves are inoperable. Many of these valves are closed resulting in a disruption of the operation and efficiency of the water utility.

In conjunction with a valve program which verifies line sizes by the operation of each valve, a mapping of the water utility should be performed and the existing maps updated. A typical gate valve requires three turns per inch in diameter to open or close the valve. For example, a six-inch valve would require 18 turns to open or close the valve. To seat the valve requires an additional two turns. Therefore, it takes approximately 20 turns to operate a six-inch valve.

SYSTEM OPERATIONS TO ELIMINATE MISCELLANEOUS LOSSES

After the meter testing and leak detection programs are completed, the system operation should be investigated to determine whether the system is functioning properly. Any altitude valves should be set to eliminate unintentional reservoir overflows. Pressure reducing valves, pressure relief valves and search control valves should be checked and set properly to insure good system operation.

The meter reading and accounting system should be evaluated to correlate the numbers of dials on a meter with the corresponding stationary zeros on the billing and accounting records. This will insure that the correct units of water are read and recorded. It is not uncommon to have different numbers of dials on same sizes of meters. Therefore, we cannot assume that the first five dials on every meter (even with the same manufacturer) can be read correctly due to the inconsistency of the standardization.

PERFORMING A RATE STUDY AND BUDGET ANALYSIS

It is estimated that the water utilities in the United States are losing approximately $2,000,000 per day or an excess of $800,000,000 per year in revenue due to improper water rate structuring. Water rate structuring for a utility is essential. The customer must be considered because the utility has the responsibility to be efficient in all aspects of their services. Before the water audit is completed and the utility has determined their most efficient operation, a water rate structure should be developed to support the total operation of the utility. The key areas are the following:

1. Operation and maintenance
2. System rehabilitation or the replacement of those appurtenances which are wearing out.
3. Capital improvements or those new facilities added to the water utility
4. Retirement of debt
5. Contribution to the general fund

It seems to be the experience of most utilities to budget for rehabilitation or replacement of the portions of the system which are wearing out. The area of capital improvements should be paid for by the revenues of the utility over long term investment of capital improvement funds.

The water utility can obtain the revenues in several ways; some of which are as follows:

1. The sale of water
2. The cost of service on a demand rate by the size of meter
3. A connection or by-in fee to the water utility (development fee)
4. Stand-by charges such as cost for fireline services and fire service demand rates.

It is recommended that a utility use a water rate structure called a "Two-Part System". A minimum monthly cost is established as the base charge and a commodity charge or cost per unit of water be charged in addition to the base rate charge.

If a utility uses wise planning in establishing their water rates structuring and they use the Two-Part System, the budgeting of the utility is considered as the rates are determined. A Two-Part Rate and commodity charge are established. On a ten-year program of rate making, a utility can give a certain amount of water which is covered by the base charge. For example, a charge of $6.00 per month allows 10,000 gallons of water use per month without additional cost.

If adequate and thorough planning is used in rate making, a period of several years can elapse before additional rate making and base charges are necessary. This is accomplished by reducing the quantity of water covered by the base rate without changing the commodity charge to establish additional revenues for the utility over the period of years. For example, a charge of $6.00 per month for the first 10,000 gallons of water would be the rate this year without additional costs. Then, if in two or three years additional funds were needed, the base rate and commodity charge could remain the same but the amount of water given is reduced from 10,000 gallons to 8,000 gallons or 5,000 depending on the additional funds needed. If it were determined that surplus funds were evident, then additional water could be given without changing the base rate or commodity charges, but only the amount of water given with the base rate charge.

A conservation rate structure can be established from the Two-Part System by using a multiple incline rate. For example, the first unit of water will cost the customer $.50 per 1,000 gallons for 10,000 gallons, then the next 5,000 gallons may cost $.60 per 1,000 gallons, the next 5,000 gallons may cost $.80 per 1,000 gallons, the next 2,000 gallons may cost $1.00 per 1,000 gallons, and so on, as needed. This type of rate structuring encourages water conservation and provides adequate revenues for a utility without penalizing the single occupant resident (typically senior citizens) with high utility rates but distributes the cost of services to those who are using large amounts of water and particularly wasting the water.

A water utility is a natural resource utility and should be self-sustaining. The utility need not look to other agencies within their city, state, or federal government for funding of operation and maintenance, rehabilitation and capital improvements.

Rate structuring is an essential part of a progressive water utility. If in the water utility industry indiscretion is used and innovative, fair, and equitable rate charges are established, water rates can be kept at a reasonable cost and the utility be self-sustaining financially. It is our challenge to be effective, efficient, and do all in our power to see that we, as operators of utilities, provide the highest efficiency as well as the highest quality of water for our customers.

AN INVESTIGATION OF THE DISTRIBUTION OF TRIHALOMETHANES WITHIN GAC CONTACTORS

by

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INTRODUCTION

Buffalo Pound Lake is the source of all of the potable water for Moose Jaw, Saskatchewan, and most of the supply for Regina. This shallow prairie lake is highly eutrophic and experiences algae blooms in most summers. Under average summer conditions the plant has historically produced a finished water with high total organic carbon (TOC) levels of 4 - 8 mg/L, appreciable trihalomethanes (80 - 150 ug/L), and noticeable taste and odour.

In 1980 a consultant's study examined many different treatment alternatives for taste and odour reduction and concluded that GAC contactors after full treatment were the best solution (CH2M Hill, 1980). However, the occurrence or removal of specific organic compounds in the water was not addressed.

Full scale GAC postcontactors were installed in 1985. These contactors follow conventional treatment, which consists of potassium permanganate addition (only during algae blooms), pre-chlorination, coagulation, flocculation and sedimentation, pH adjustment, and mixed media filtration. The plant capacity is 136 ML/day.

During 1982 a pilot study examined removal of trihalomethanes from finished water using GAC (Andrews et al., 1983). It confirmed previous findings by Forsyth et al (1982), that significantly higher carbon loadings were achieved in the top segment of a GAC bed. The investigation described in this paper was conducted during 1984 to examine specifically the loading distribution of trihalomethanes and total organic carbon in the context of assessing different carbon types for full scale use.

BACKGROUND

A number of investigations have examined the ability of GAC to remove trihalomethanes. Representative results are described below, with emphasis on trihalomethane distribution in GAC beds.

In pilot scale studies at Philadelphia, Yohe et al. (1981) observed sharp breakthrough of chloroform and chloroform precursors immediately following startup in columns of 1 m bed depth. Cumulative mass loadings of trihalomethanes on the carbon displayed a levelling trend as exhaustion was approached (Yohe et al., 1981; Cairo et al. 1979).

Cairo et al. (1979) evaluated the performance of carbon on the basis of mass loadings. Comparing different modes of operation, chloroform removal was 20% higher in a contractor than in a filter adsorber (where existing sand filters were modified to accept GAC media). The carbon in both columns was Filtrasorb® 300. In the same study, a carbon with smaller grain size (Filtrasorb® 400) was found to be more effective in removing the mass loadings of chloroform than a larger carbon (Filtrasorb® 300) when both were operated in a similar contactor mode. Forsyth et al. (1982) reviewed chloroform cumulative mass loadings and found that loadings in the range of 0.42 to 1.6 mg/g of GAC would be expected at column exhaustion.

An examination of mass loading with respect to depth was presented by the USEPA (1980). Expressed on a per gram of GAC basis chloroform loadings increased with depth, whereas bromodichloromethane loadings did not appear to change significantly. Average chloroform and bromodichloromethane influent concentrations were 67 ug/L and 47 ug/L respectively.

Alben and Sphirt (1983) examined distribution profiles for various volatile organics and organic acids on GAC using four pilot columns operated in series. Monitoring ports were spaced at intervals of 120 cm along the bed depth, i.e. after each column. From the profiles it was possible to construct a case history for GAC saturation and evaluate chromatographic displacement within the pilot scale bed. Although localized depletion of chloroform on the GAC was noted at the inlet of the system after 26 weeks of operation, other compounds less volatile than chloroform were noted to be concentrated in this same region. Cumulative adsorption profiles of TTHM were found to closely resemble chloroform trends.

Size classification data for particles in a GAC adsorber have been reported by Alben and Sphirt (1981) and Alben et al. (1983). In these studies the smallest particles were present at the top of the bed whereas larger particles were found in lower segments. This classical distribution caused by backwashing and its effects on adsorption have been reviewed by others (Neely and Isacoff, 1982; Weber, 1972; Weber et al., 1983). The adsorptive capacity of a nonporous adsorbent has been found to vary inversely with particle diameter (Weber, 1972). In GAC studies adsorption rates decrease with increasing particle size, resulting in distortion of the mass transfer zone (Neely and Isacoff, 1982).

Experiments by Zogorski and Faust (1978) evaluated GAC columns containing various carbon particle sizes. These studies revealed that the rate of movement of the mass transfer zone using 2,4-dichlorophenol as the adsorbate was independent of the size of the adsorbent. However, the relative steepness of the breakthrough curve was a direct result of different rates of adsorption onto the various sizes of activated carbon: the larger the carbon size, the broader the breakthrough curve. Similar studies conducted by Rosene et al. (1980) compared breakthrough profiles of chloroform at influent concentrations of approximately 200 ug/L using coarse and fine GAC. A much broader breakthrough curve was obtained with the coarse carbon, indicating slower kinetics and a wider mass transfer zone.

A study by Mullins et al. (1980) suggests that TTHM adsorption capacity at equilibrium should not be greatly influenced by particle size since trihalomethanes are small molecules and would have access to most pores. However, broader breakthrough curves and wider mass transfer zones may be obtained from larger particles as opposed to smaller particles.

Previous pilot studies conducted at the Buffalo Pound Water Treatment Plant have shown that appreciable THM loadings occur in the uppermost segments of GAC beds (Andrews et al, 1983; Forsyth et al., 1982). For this reason a partial bed replacement was investigated to assess its effectiveness on THM removal efficiency and the possibility of extending bed life.

OBJECTIVES

The complete 88 day study conducted at Buffalo Pound (Andrews, 1986) examined in detail the distribution of adsorbed trihalomethanes and TOC in pilot scale GAC columns. This paper addresses the following specific objectives:

1. to determine the loading distribution of trihalomethanes in pilot scale GAC columns of varying depths and carbon type,

2. to assess the effectiveness of a partial bed replacement on trihalomethane removal efficiency and loading characteristics,

3. to evaluate the effect of carbon grain size on trihalomethane adsorption, and,

4. to determine the usefulness of adsorption isotherms in predicting column loading behaviour.

METHODS AND PROCEDURES

A summary of the principle methods and procedures is given in this section. Complete details are provided by Andrews (1986).

Adsorption Isotherms

Isotherms were conducted using carbon pulverized with an Oster blender. The carbon was pre-wetted, and for the test sufficient sodium thiosulfate was added to destroy up to 6 mg/L of free chlorine. The bottles were filled head-space free, sealed and agitated for 24 hours using a Labconco roller apparatus. The 24 hour equilibration time was based upon equipment limitations and previous kinetic investigations by Andrews et al. (1983) which had shown that equilibrium for the major THMs was reached after 6 hours when using 30 x 40 mesh GAC. When using pulverized GAC, stated equilibrium times range from 2 hours (Fochtman and Dobbs, 1980) to 4 to 7 days (Crittenden and Hand, 1983). The combination of a minimum 42 hour wetting period followed by 24 hours of equilibration at the time was felt to address these requirements satisfactorily. However, later data analysis suggested that the isotherms may not have actually reached equilibrium.

Pilot Scale GAC Columns

The pilot columns used in this study were 2.4 m long by 15 cm inside diameter. The columns were of PVC material chosen for ease of handling and availability. Studies by Kreff et al. (1981) suggest that pilot scale columns constructed of PVC have an insignificant effect on levels of purgeable volatile organic compounds. Each of three identical columns (A, B, and CA) was filled to a depth of 153 cm with fresh carbon (Table 1). A fourth column designated "CB" was connected in series to receive the effluent of Column CA. This column was filled to the same depth as the others with the same carbon as in Column CA. A fifth column which was considerably shorter than the others (bed depth 31 cm) was designated "S". This column served as an isolated shallow bed allowing direct comparisons to be made to the top portion of a deeper bed (Column CA).

Table 1

Granular activated carbons used in study

Carbon Name	Norit® ROW 0.8 (Column A)	Witco® 950 (Column B)	Filtrasorb® 300 (Columns CA, CB & S)
Manufacturer	Norit	Witco	Calgon
Mesh	NA	8 x 30	8 x 30
Wetted Density (kg/m^3)	355	460-530	420
Surface Area (m^2/g)	NA	1000-1100	1000
Iodine No.	900	1050	900
Abrasion No. (Ro-Tap)	NA	NA	70-80
Ash Content (%)	6	1	8
Average Pore Size (m x 10^{-9})	NA	0.5-3.0	1.0-2.0
Source	Peat	Petroleum	Bituminous Coal

NA - Not Available

The flowrate of finished plant chlorinated water to each column was 3.5 mm/s. This value approximated normal sand filter throughput in summer months and was the design flowrate of the full scale GAC contactors. This flow provided an empty bed contact time of 7.3 minutes for Columns A, B, CA, CB, and 1.5 minutes for the shallower Column S.

Adsorption profiles were obtained by the use of stainless steel sampling ports located at 10.3 cm (4 in.) intervals down the side of each column. For example, ports designated as #1 and #3 represent bed depths of 10 cm and 31 cm respectively. The ports were 7.5 mm ID and projected 5 cm into the column. Stainless steel screens were incorporated into the ports to exclude carbon. Effluent samples for Columns A, B, CA and CB were collected near the bottom of the bed ports designated A15, B15, CA15, and CB13 respectively. Plant finished water from the same source as column influent served as backwash water. The backwash procedure lasted approximately 10 minutes with flowrates varying between 3.6 mm/s and 10.8 mm/s. Samples of backwash effluent were collected routinely for visual examination to determine the type of suspended matter present.

At day 81 of the study the top 31 cm of carbon was removed from Column CA and replaced with fresh GAC to observe the effects of a 20% bed replacement. The complete 31 cm bed in Column S was replaced at the same time.

Analytical

Chloroform, bromodichloromethane, and dibromochloromethane concentrations were determined by gas chromatrography on a Hewlett-Packard 5840A unit equipped with an electron capture detector. Sample volumes of 5 microliters were introduced by direct aqueous injection (DAI) into a 1.8 m by 6 mm ID glass column, packed with Chromosorb 101. A carrier gas mixture of 5% methane in argon was used at a flowrate of 32.2 mL/min. The design of this study required that a large number of samples be analyzed daily on a routine basis. This precluded use of the more time consuming purge-and-trap method and dictated use of the DAI method normally employed by the Buffalo Pound plant. This method may be expected to yield concentrations higher than the purge-and-trap method because of its measurement of both actual and potential trihalomethanes. However, in the context of this study, an increase of this type was not considered significant since most results were used for comparative purposes among and within the various carbon columns.

Total trihalomethanes (TTHM) are expressed as the sum of chloroform, bromodichloromethane and dibromochloromethane. Bromoform was typically less than 1 ug/L, or approximately 1% of the total.

Other parameters (TOC, chlorine residuals, etc.) were measured as described by Andrews (1986).

Data Analysis

All data for monitored compounds were input to a computer program called GAC7 written specifically for this study (Andrews, 1986). For each column segment the program was capable of calculating loadings and percentage removals, and estimating breakthrough times. The calculations could be readily updated as new data became available.

Loading Calculations

Exhaustion or "saturation" as defined by some workers is usually assumed to occur when the effluent concentration reaches some preselected fraction of the influent concentration. In a water treatment plant, the influent concentration or a particular compound will usually vary continually and may follow seasonal trends. Since adsorption is an equilibrium process an increase or decrease in adsorption capacity will result from an increase or decrease respectively in influent concentration. The term "apparent exhaustion" was therefore used as an operational definition in this study and represented the point where the effluent concentration had either reached or paralleled the average influent concentration. The exact procedure used to determine the time of apparent exhaustion is described by Andrews (1986).

Loadings at apparent exhaustion were calculated by dividing the net weight of a compound deposited in a column segment by the weight of carbon in that segment. Loadings were calculated for each sampling interval and summed over the 88 day operating period. Thus a value for chloroform in segment 0-1 would represent the difference between the column influent concentration and the concentration at Port 1 multiplied by the flow to the column and divided by the weight of carbon between the top of the column and Port 1. For segments which had not reached apparent exhaustion during the test, an extrapolated amount deposited after the end of the test was estimated by graphical integration assuming a constant influent concentration (Andrews, 1986).

Loadings are thus based on differences in liquid phase concentrations; to actually measure them would have been extremely difficult. Loading calculations assume no generation of THMs within the column.

RESULTS

Only the results for total trihalomethanes (TTHMs) are discussed in this paper. Results for the individual trihalomethane components are presented in detail by Andrews (1986). The behaviour of chloroform, the major trihalomethane component, largely determined the pattern for TTHMs. Although a detailed analysis of TOC adsorption was also conducted, these results are only included here where they are necessary to an interpretation of the TTHM results.

Removal in Column A

Influent, effluent, and intermediate port adsorption data for TTHMs in Column A appear in Figures 1 and 2. Adsorption in Column B was slightly higher than in Column A but followed generally similar trends. The influent concentration during the 88 day study period averaged 92 ug/L and showed an increasing trend with time. Breakthrough occurred in all column segments within the first few days, as expected. Table 2 gives the time to reach apparent exhaustion for the various segments of Columns A and B. As is evident, apparent exhaustion had occurred in the lowest third of Column A (segment 10-15) by day 77, corresponding to 15,100 bed volumes (BV).

Figure 1

TTHM in finished water and removal by 10 and 31 cm GAC (Column A)

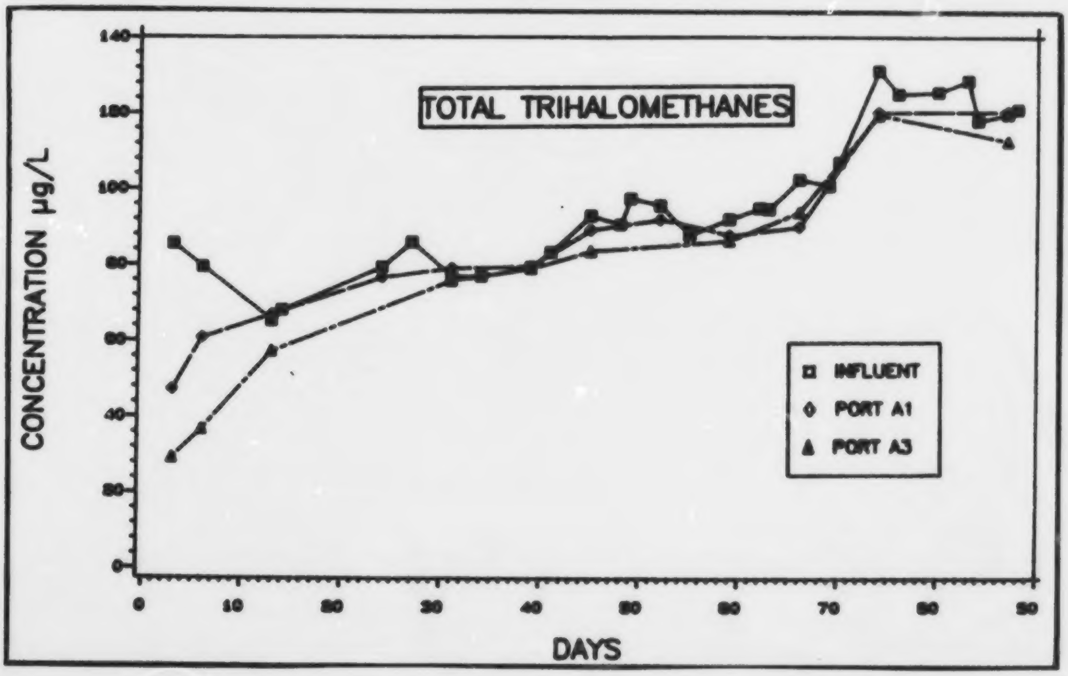


Figure 2

TTHM in finished water and removal by
51, 102 and 153 cm GAC (Column A)

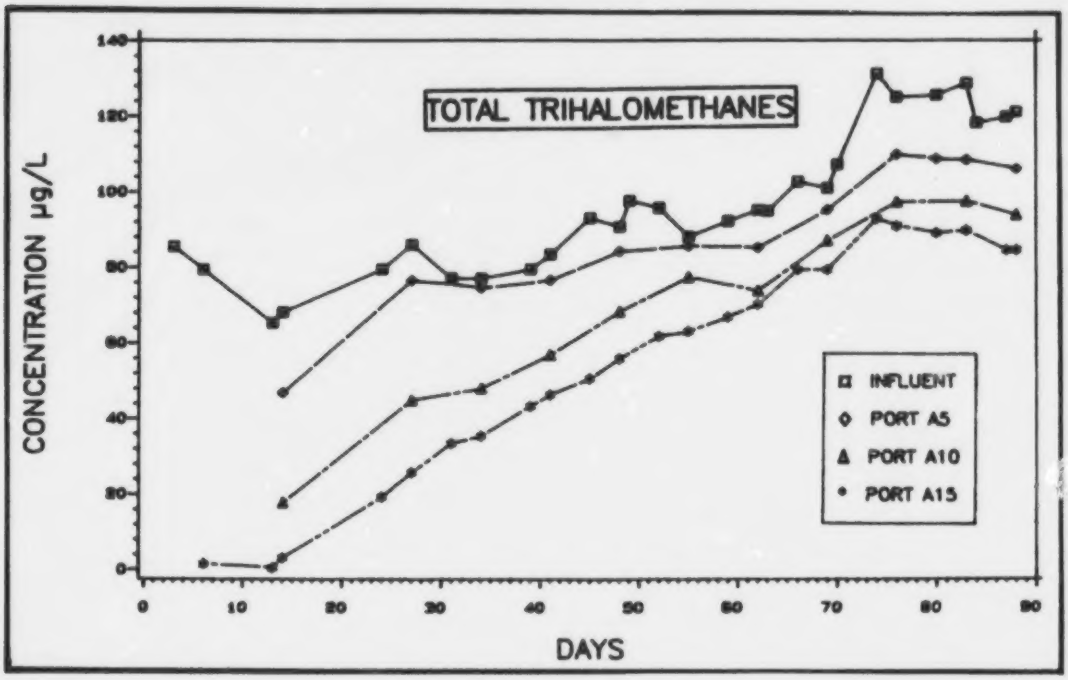


Table 2

Time to reach apparent exhaustion

	Time of Apparent Exhaustion (days)	
Segment	Column A	Column B
0-1	50.0	54.9
1-3	58.4	54.5
0-5	64.1	63.1
5-10	72.0	86.8
10-15	76.7	131.6*

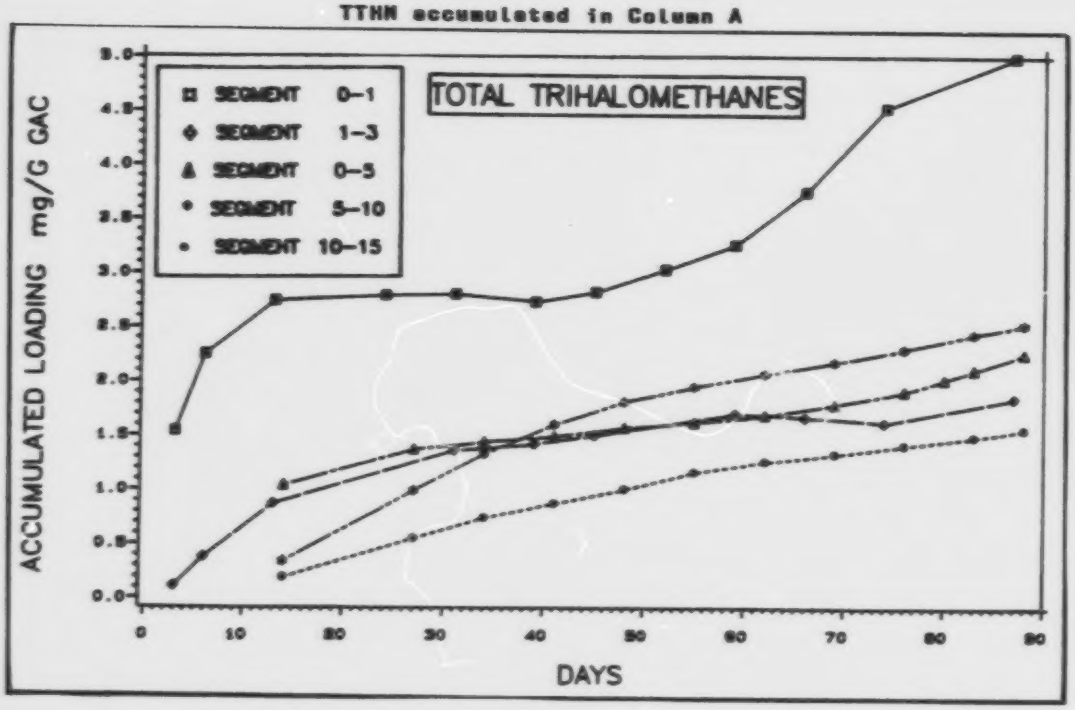
Average influent concentration: 92 ug/L
Test duration: 88 days
*Extrapolated value

The adsorptive capacity at apparent exhaustion was the highest in segment 0-1 at 2.87 mg/g GAC (Figure 3). Segment 1-3 immediately below displayed a much lower loading at 1.70 mg/g GAC, and seemed unlikely to approach the segment 0-1 value. Considering the column as three equal segments (0-5, 5-10 and 10-15) the highest loading (2.23 mg/g GAC) at apparent exhaustion occurred in segment

5-10. Loadings obtained at the end of the test period increased in all segments as a direct result of the increasing influent concentration.

Figure 3

TTHM accumulated in Column A



Immediately following startup, loading of the carbon in the upper-most segments appears very rapid (Figure 3). Once a levelling, albeit moderately increasing trend had been established (days 14 to 40, Figure 1) a similar levelling trend or decreased rate of loading was noted in segment 0-1. Apparent exhaustion immediately followed this period for segments 0-1 and 1-3 as seen in Figure 1. As the influent concentration increased from day 45 to the end of the study period, the loading in segment 0-1 increased. The loading in segment 0-5 showed a slow increase with time and appeared inhibited to respond directly. The slow loading trend of segment 0-5 was accompanied by the increase in adsorptive capacity of segment 5-10 following day 34. The loading in this middle segment is actually higher than in the upper third of the bed. The distance between the accumulated loading lines of segments 5-10 and 10-15 increased with an increasing influent concentration due to the high adsorptive capacity of the middle segment.

Once apparent exhaustion had been reached for Ports A1 and A3 (Figure 1), the TTHM levels at these depths roughly paralleled the fluctuations in the influent as expected. At Ports A5, A10 and A15 plateaus were reached near the end of the study period as the influent concentration levelled off.

Of particular importance is the significantly higher adsorptive capacity of segment 0-1. In this segment 7% of the bed adsorbed 16% of the TTHM entering the column during the test period even though the whole column had reached apparent exhaustion before the end of this time.

Evaluation of Partial Bed Replacement

Columns CA and CB were used to study both the effect of a partial bed replacement after 61 days of operation, and the effect of an extended bed depth. Column S examined the effect of the complete replacement of a shallow bed of the same carbon.

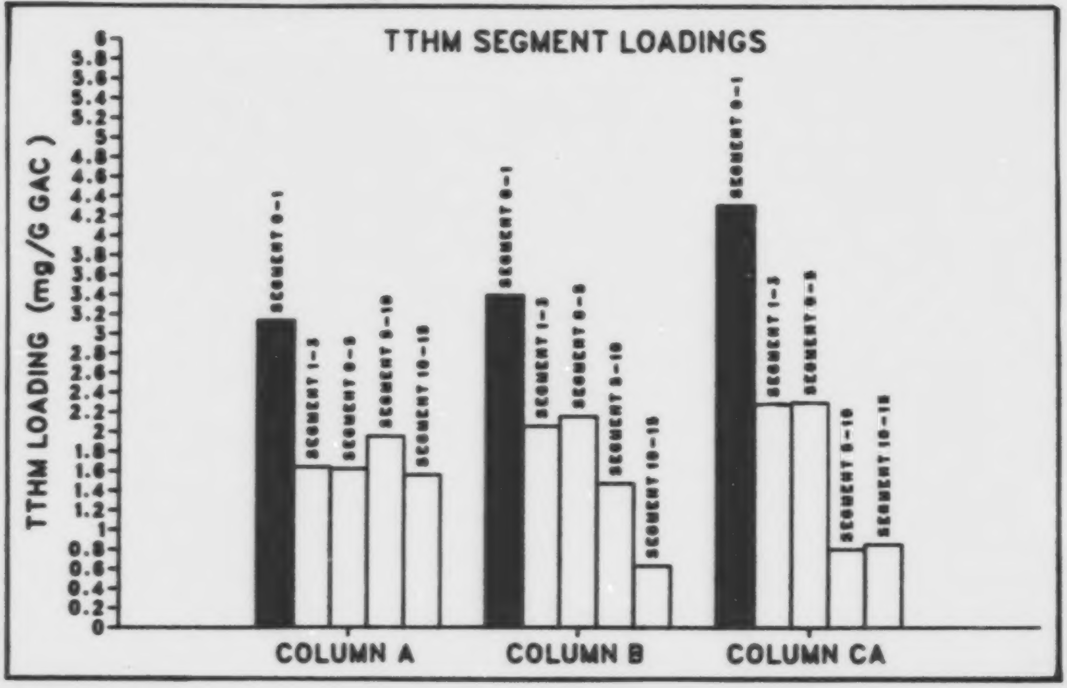
TTHM loadings for Column CA at day 61 are compared to those for Columns A and B in Figure 4.

Loadings in segments 0-1, 1-3 and 0-5 of Column CA were slightly higher than for either Column A or B, but in similar proportion to each other. Loadings in segments 5-10 and 10-15 were different in either proportion or magnitude from the other two columns.

These results confirmed previous pilot studies [Forsyth et al. 1982; Andrews et al., 1983] which showed that Filtrasorb® 300 [Column CA] removed TTHMs more effectively than the other carbons in upper bed segments. It was for this reason that Filtrasorb® 300 had been chosen for bed replacement studies.

Figure 4

Segment loadings in Columns A, B and CA at day 61



At day 61 the top 31 cm of carbon [segment 0-3] was removed from Columns CA and S, and replaced with fresh GAC. This represented 20% and 100% of the carbon present in Columns Ca and S respectively and permitted exploring the possibility of increasing bed life with only a partial bed replacement. However, an important difference existed between the two beds. In Column CA, the finest 20% of the bed had been removed and replaced with material covering the complete size range. Following backwashing, this new material would be distributed throughout the entire bed depth, bringing partially exhausted finer material into the upper segments.

TTHM concentration profiles for Columns CA and S are shown in Figures 5 and 6 respectively. Large decreases in TTHM concentrations were noted at Port 3 immediately following the carbon replacement, as expected.

Figure 5

TTHM in finished water and removal by 10 and 31 cm GAC (Column CA)

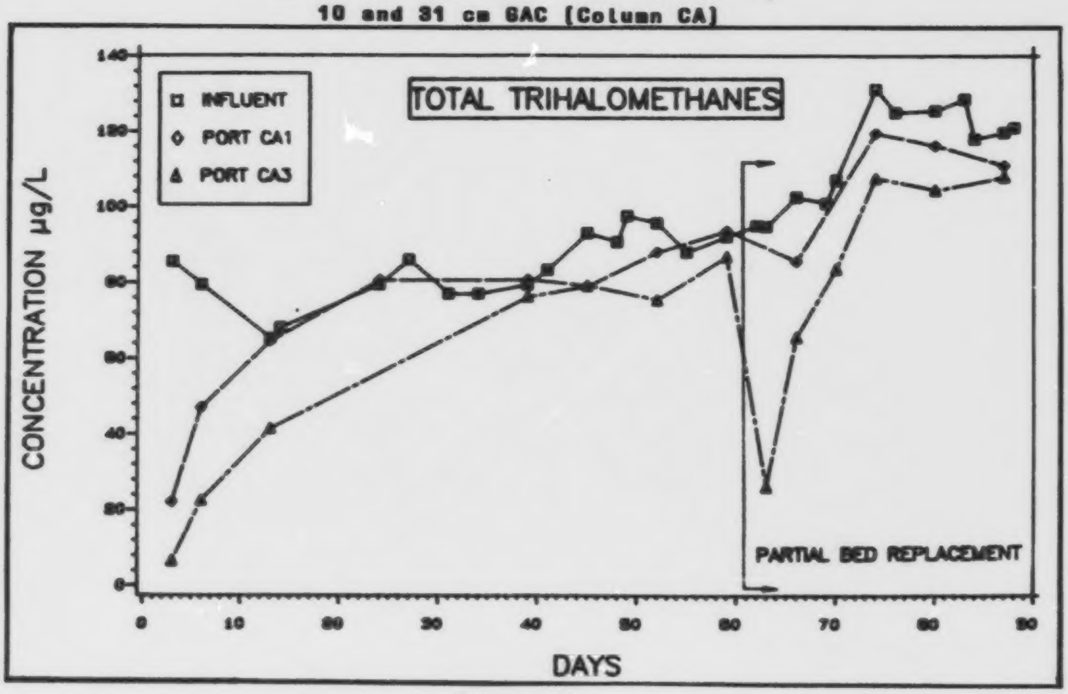
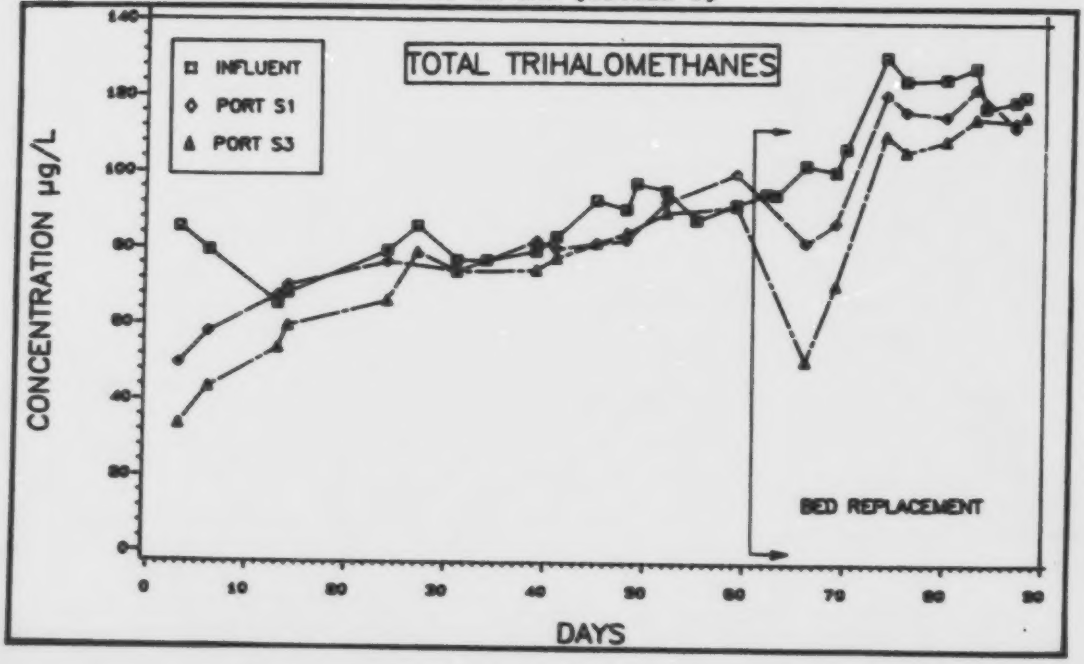


Figure 6

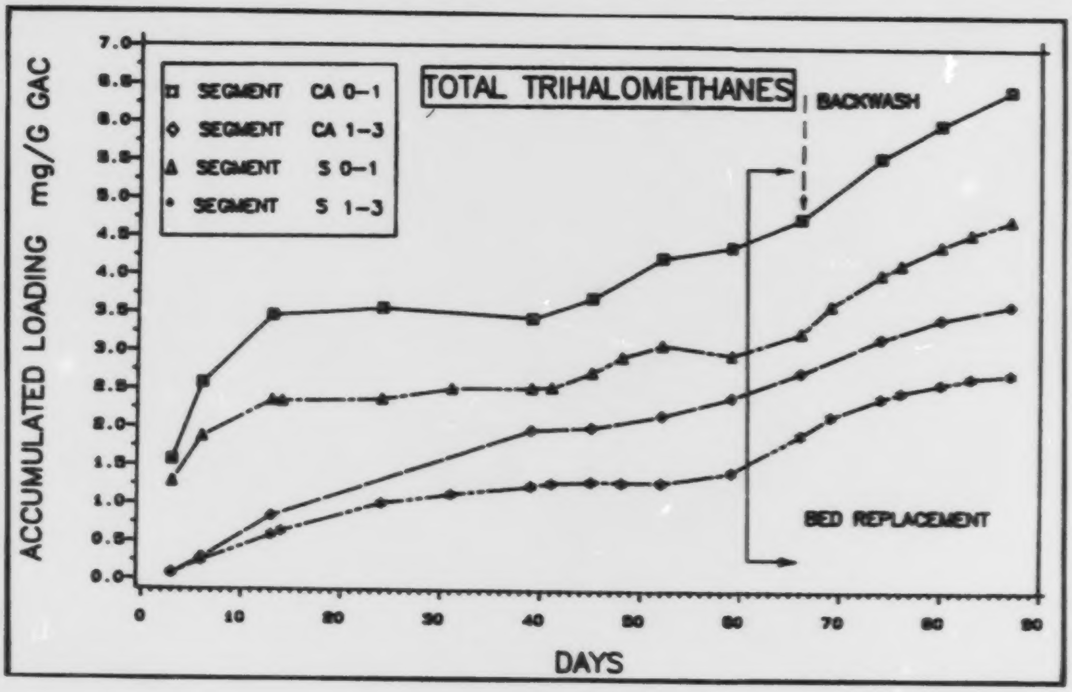
TTHM in finished water and removal by 10 and 31 cm GAC (Column S)



A comparison of accumulated TTHMs at Ports CA1, CA3, S1 and S3 appears in Figure 7. Prior to the carbon replacement, loadings in the shallow bed were significantly lower than in the full depth bed: segment S 0-1 had accumulated only 68% as much TTHM as the corresponding CA 0-1 segment, while segment S 1-3 had accumulated only 60% as much as segment CA 1-3. These values illustrate the significantly enhanced adsorption effect in upper segments of columns attributable to bed stratification, which was visually apparent in Column CA. Higher adsorptive capacity would be expected in the upper segments of the deeper bed CA, due to the smaller particle size in that region.

Figure 7

TTHM accumulated in Columns CA and S



Immediately following the carbon replacement an increase in the rate of accumulation is evident for both Columns CA and S. After backwashing at day 66, accumulation profiles of segments CA 0-1 and S 0-1 appear roughly parallel. However, by the end of the test period (27 days following bed replacement) segments S 0-1 and S 1-3 had accumulated 88% and 106% respectively of the loading in segment CA 0-1 and CA 1-3 after the replacement. The lower loading in segment S 0-1 suggests that the effect of fresh carbon did not fully compensate for the smaller particle size available in segment CA 0-1. The results for segments S 1-3 and CA 1-3 are very close, but suggest a slight advantage due to the fresh carbon in the shorter column.

The total loading of Column S is compared to the Column CA segment 0-3, representing the same amount of carbon, in Table 3. The additional loading to the end of the study was very similar on an absolute basis in both columns.

Table 3

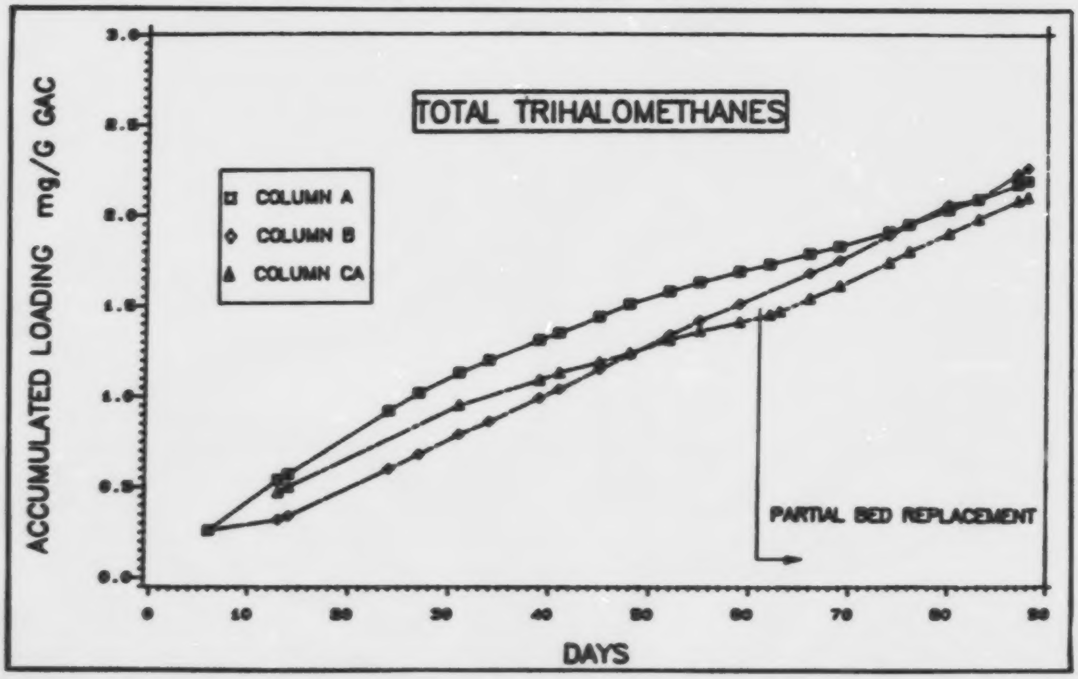
TTHM segment loadings before and after bed replacement - Columns CA and S

	Loading (mg/g GAC) - Segment 0-3	
Prior to Partial Bed Replacement	1.95	3.07
End of Study	3.40	4.57
Increase (mg/g)	1.45	1.50

The ability of an increasing influent concentration to cause increased loadings was examined by comparing cumulative TTHM loadings (segment 1-15) for Columns A, B and CA on the same graph (Figure 8). Adsorptive capacities in Columns A and B appear to be unaffected by increased influent TTHM concentrations following day 63. On a more localized scale, cumulative TTHM loadings for segments 0-1 and 1-3 in Column A were presented in Figure 3. No concentration effect was noted for segment 1-3 in Column A, however segment 0-1 did exhibit a slight loading rate increase following day 63. It may therefore be concluded that the increase in TTHM loading of column CA following carbon replacement was a direct result of the new unused capacity except possibly in segment 0-1 where concentration may induce a small capacity increase.

Figure 8

TTHM accumulated in Columns A, B and CA



Total loadings in Columns A, B and CA are shown in Table 4 at day 59 (immediately before the replacement) and at the end of the study period. Column CA loading increased 49% following the replacement compared to 30% and 49% increases in Columns A and B which did not receive new carbon. The 19 percentage point increase in loading capacity in Column CA in excess of Column A may be attributed to the 20% bed replacement. Column B displayed no signs of reaching exhaustion and at all times removed more TTHMs than either Column A or CA.

Table 4

TTHM segment loadings before and after
bed replacement -
Columns A, B and CA

	Loadings (mg/g GAC) - Total Column		
	Column A	Column B	Column CA
Prior to Partial Bed Replacement in Column CA	1.69	1.51	1.41
End of Study	2.19	2.26	2.10
Increase (mg/g)	0.50	0.75	0.69
(%)	30	49	49

Removal of TTHMs by Deep Beds - Columns CA & CB

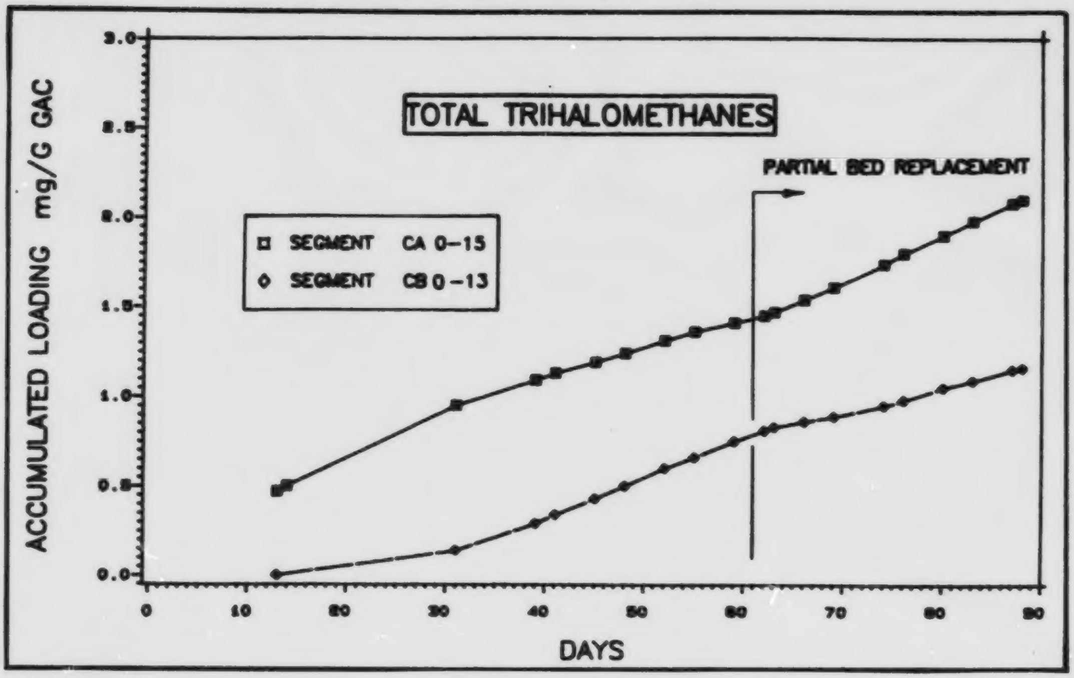
The series combination of Columns CA and CB allowed adsorption of TTHMs to be studied in a bed depth totalling 286 cm. Actual carbon depth in Column CB measured 153 cm as in Column CA but the effluent was monitored at the lowest port (Port 13), representing a bed depth of 133 cm.

TTHM accumulation in Columns CA and CB is shown in Figure 9. At the end of the test period loadings of 2.10 mg/g GAC and 1.16 mg/g GAC were present in Columns CA and CB respectively. The effect of the 20% carbon replacement in Column CA at day 61 is noted as an increase in slope of the Column CA accumulation curve and decrease in slope of the Column CB accumulation curve. These deviations suggest that the increase in upper bed capacity resulted in a lower influent concentration to Column CB and consequently a decreased loading rate. The assumption that this behaviour occurred prior to complete apparent exhaustion of either Column CA or CB was confirmed by the positive slope noted in both accumulation profiles.

If Column CA and CB had been one single bed, the observed behaviour would have been somewhat different. The greater depth of fine particles at the top of the bed would have produced an even greater enhancement of adsorption in this region.

Figure 9

TTHM accumulated in Columns CA and CB



Total Organic Carbon (TOC)

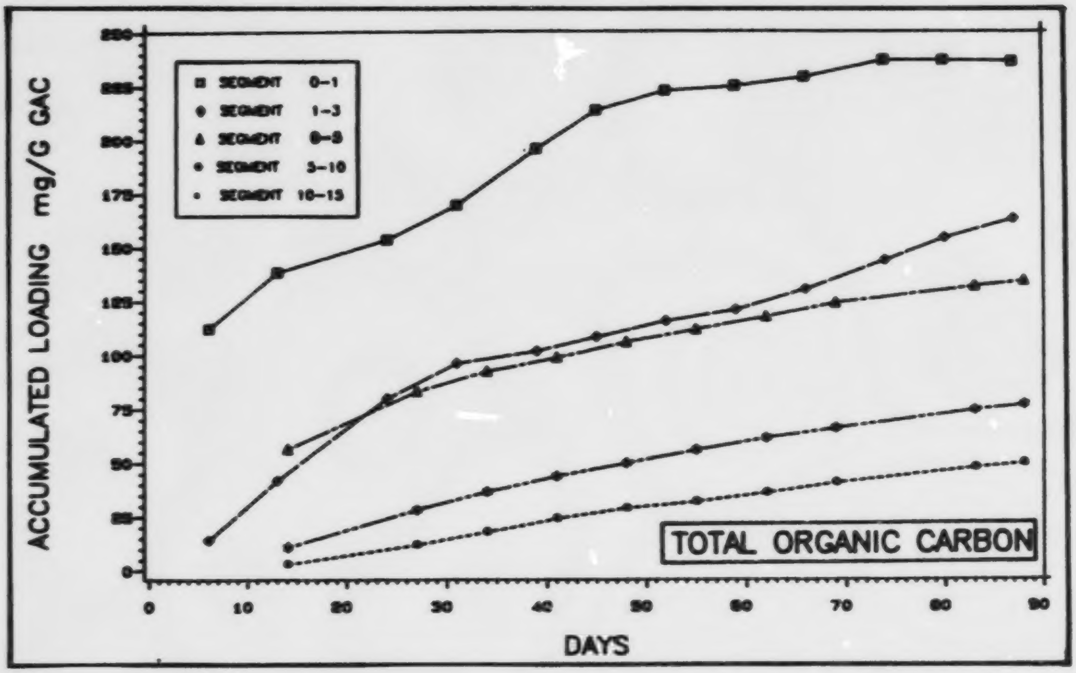
The adsorption of TOC was assessed in detail by Andrews (1986). Representative results are presented here only to support the discussion concerning TTHMs.

TOC data did not exhibit the characteristic breakthrough curves noted for trihalomethanes. Instead, three well defined phases of adsorption existed. These phases, as identified by McGuire et al. (1983) are: (1) immediate low level leakage, (2) a regular orderly breakthrough, and (3) a pseudo-steady-state plateau. The last phase may be due to biological removal or slow adsorption kinetics (Maloney et al., 1983).

The influent TOC had an average value of 3.6 mg/L and varied between 3.3 and 4.1 mg/L. There was a slight decreasing trend during the latter half of the study period. Figure 10 shows TOC loadings for Column A. It is evident that the upper column segments had a much higher capacity than either segments 5-10 or 10-15. The lower segments would appear unlikely ever to achieve those higher loadings, based on the trends in the figure. Since the top portion of the bed adsorbed higher amounts of both TTHM and TOC, increased TTHM loadings cannot be attributed to a lower adsorption of background organics in this region.

Figure 10

TOC accumulated in Column A



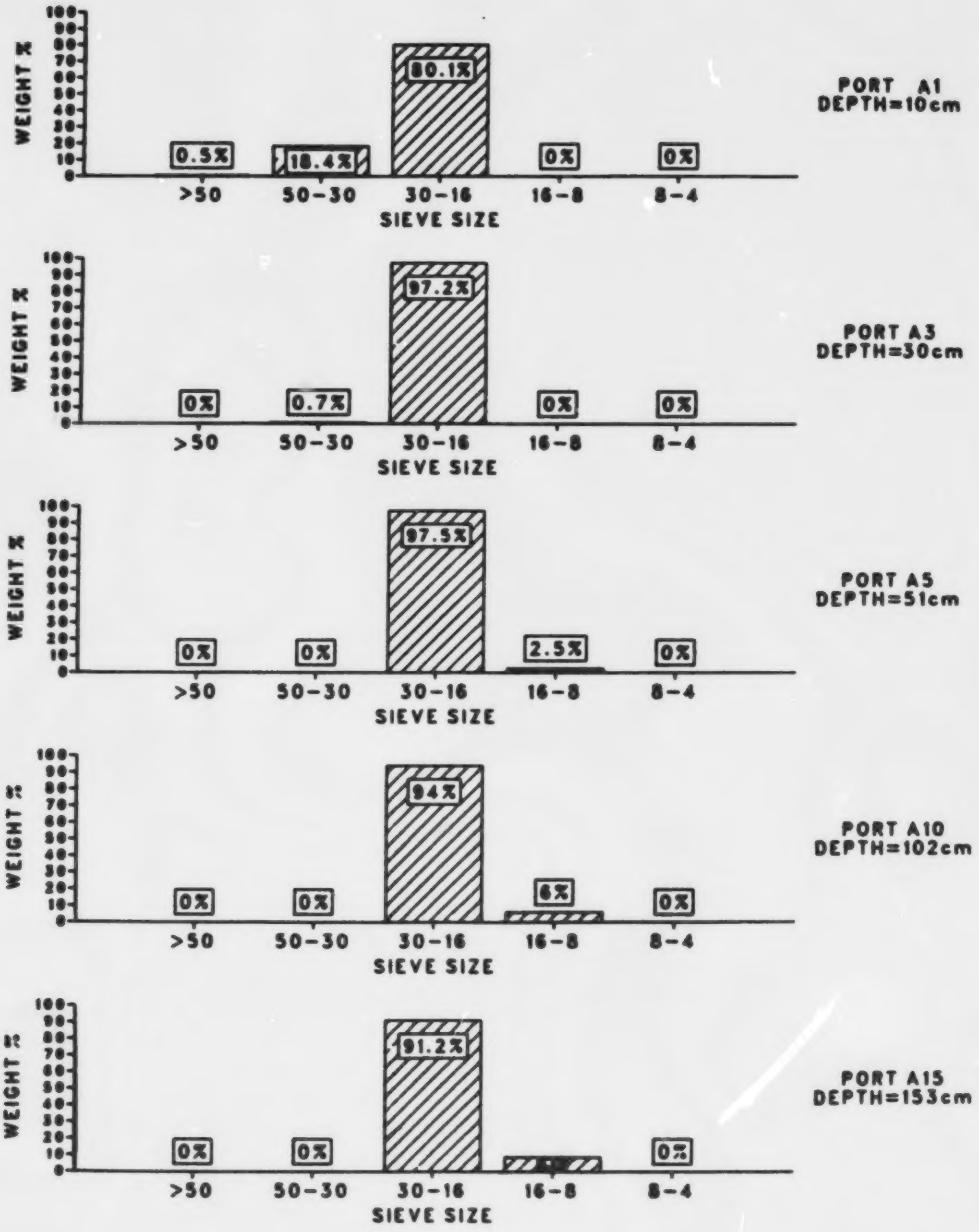
Effect of Particle Size Distribution on TTHM Adsorption

Grain size distribution data for carbon samples extracted from Column A at Ports 1, 3, 5, 10 and 15 following 14 weeks of operation appear in Figure 11. To determine if a relationship existed between TTHM loadings at apparent exhaustion and particle size distribution an SPSS^X multiple stepwise regression routine was applied [Norusis, 1983]. The percentages of carbon retained between given U.S. Standard sieve sizes were used as the independent variables. TTHM served as the dependent variable. For example, the TTHM loadings measured in segments 0-1, 1-3, 0-5, 5-10 and 10-15 were regressed upon the percentage by weight of carbon particles in the >50 sieve size range from samples collected at Ports 1, 3, 5, 10 and 15. This procedure was repeated for all size ranges. The importance of a particular grain size is reflected by the relative correlation between the dependent and independent variables. The F test at $\alpha = 0.05$ was applied to interpret regression significance.

The results are discussed only for Column A. Results for the other columns were generally similar and are presented by Andrews [1986].

Figure 11

GAC particle size distribution - Column A



Correlations between TTHM loadings and the percentage material in a given size range are shown in Table 5. A high positive correlation was established for sieve sizes smaller than 30 mesh. A negative correlation for the larger grain sizes suggests an inverse relationship between grain size and TTHM loadings. Similar trends were observed for TOC. These results agree with those of other workers (e.g. Weber, 1972), and provide support for the inferences regarding particle size made earlier. The actual correlations should be viewed in a qualitative sense only, as they are based on rather narrow variations in the percentages of carbon in each size range (Figure 11).

Table 5

Correlation of TTHM loading to particle size distribution in Column A

Correlation Coefficient	Sieve Size	Mean Particle Diameter (mm)	Significance at $\alpha = 0.05$
0.879	>50	<0.42	Yes (Slightly)
0.876	50-30	0.42	No
-0.786	30-16	0.84	No
-0.482	16-8	1.67	No
NC	8-4	3.35	No

NC - Correlation could not be computed

TTHM and TOC loadings were also regressed on functions of mean particle diameter in each segment. For TTHMs, the regression coefficients were not significantly different from zero. For TOC, significant inverse relationships were observed for diameter, diameter squared and diameter cubed (Andrews, 1986).

Comparison of Isotherm and Column Results

Isotherm results for TTHMs for the three carbons appear in Figure 12. Isotherm data for individual TTHM components and TOC, and Freundlich parameters for all isotherms are given by Andrews (1986).

A comparison of loadings obtained from isotherms and from the columns at apparent exhaustion is presented in Figure 13. Such a comparison allows the usefulness of isotherm results in predicting column behaviour to be evaluated. Loading data are complete for Columns A and B since apparent exhaustion either occurred within the study period or could be extrapolated for each of the column segments. Complete data could not be obtained for Column CA since the carbon replacement at day 61 occurred prior to lower segment apparent exhaustion. Loadings calculated by extrapolation for these segments would be meaningless since the effect of the replacement on loading behaviour could not be accurately defined.

Column A displayed an extremely good agreement between isotherm and loading data for segments 1-3, 0-5 and 0-15. However the 0-1 segment loading was 1.7 times the isotherm value. Loadings which exceeded isotherm predictions were also detected at mid-column (segment 5-10).

Figure 12

TTHM adsorption isotherms — Norit® ROW 0.8, Witco® 950, and Filtrasorb® 300

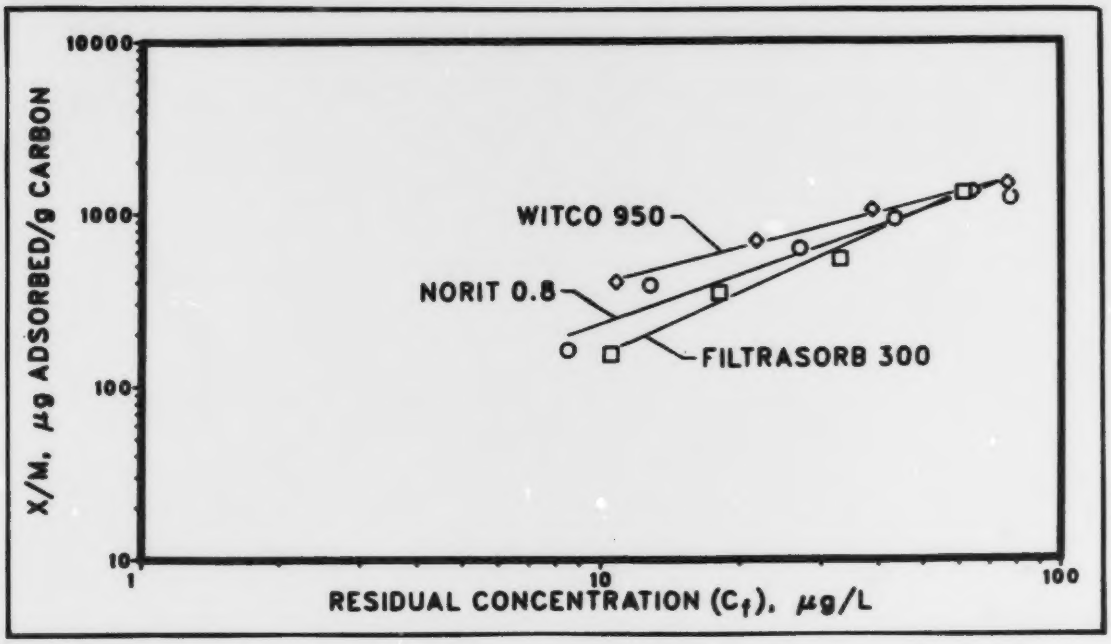
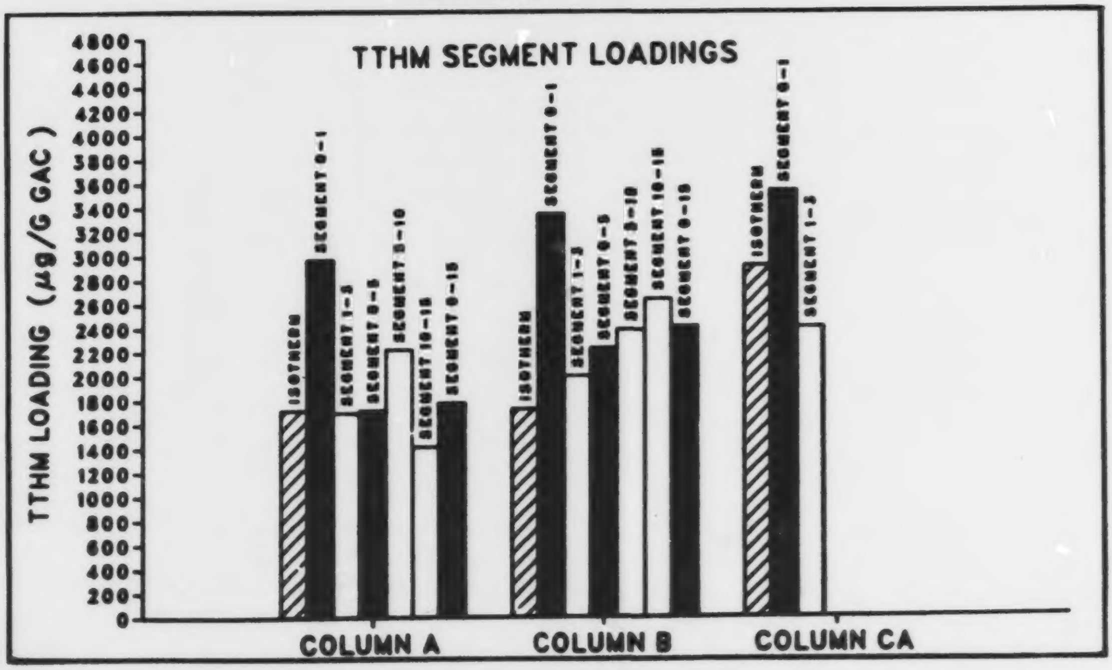


Figure 13

Comparison of isotherm and column TTHM loadings — Columns A, B and CA



For Column B, segment 1-3 provided the best correlation between isotherm and column loading data. Although the isotherm values for the two carbons were similar, actual loadings for all column segments exceeded those for column A. Again, segment 0-1 showed a significantly higher loading than the isotherm value.

As noted earlier, Column CA loading data are limited to only the uppermost segments as a result of the carbon replacement at day 61. The isotherm value was however a reasonable predictor of upper column behaviour.

TOC column loadings were not well correlated to isotherm data for Columns A, B or CA. In general the isotherms predicted the loadings observed in mid- to lower column depths. Upper segment loadings exceeded isotherm values by an even greater percentage than for TTHM.

There are several possible explanations for the discrepancy between isotherm and upper column segment results. The most likely is that the isotherms did not come to equilibrium. Crittenden and Hand (1983) has suggested that trihalomethane adsorption in the presence of background organics requires especially long equilibrium times. Particle size can be ruled out: the isotherms should have given higher rather than lower loadings since the carbon was pulverized. Biological regeneration in the columns is considered unlikely for TTHMs, especially in the presence of a free chlorine residual in the uppermost column segment. If biological activity were to play a role it would be lower in the column. Ninety percent of the free chlorine was eliminated by Port 1, although the combined residual persisted lower in the column.

It is possible that the chlorine may have altered the surface properties of the carbon in segment 0-1, making it more favorable for TTHM adsorption. Chlorine was not present in the isotherm tests, as the samples were quenched. However, the difference between segments S 0-1 and CA 0-1 (Figure 7), both containing the same carbon and both receiving chlorine, is much greater than that between segment CA 0-1 and the isotherm (Figure 13). This suggests that if chlorine is a factor it is of lesser importance than particle size.

While there is some lack of fit to the Freundlich equation evident in the isotherm data, the error from this source is considered minimal. The loadings are calculated for the average TTHM influent concentration, 92 ug/L, which is very close to the actual experimental range (Figure 12).

It is possible that some trihalomethane formation occurred in the columns. This could occur by reaction of intermediates, and would not require chlorine to be present. However, this would result in decreased rather than increased loadings when these are calculated by difference of liquid phase concentrations as in this study.

CONCLUSIONS

The following conclusions may be drawn with respect to TTHM adsorption in pilot scale GAC contactors treating finished drinking water at Buffalo Pound, Saskatchewan:

1. TTHM loadings on the carbon were significantly higher in the uppermost 10 cm of the column than in the rest of the bed. The enhanced adsorption is correlated with finer size carbon in the upper region.

2. Loadings in the top 10 cm significantly exceeded isotherm values obtained with powdered carbon. This is attributed to the fact that the isotherms did not come to equilibrium. The isotherms, as conducted, did however provide reasonable estimates of overall column TTHM loadings.

3. Similar adsorption behaviour was seen for TOC, showing that the enhanced TTHM adsorption cannot be attributed to reduced competition from background organics at the top of the column.

4. Replacement of the top portion of a carbon bed produced increased adsorption, as anticipated. However, the effect is less than expected because the average particle size of the new carbon is greater than that of the material removed.

ACKNOWLEDGEMENTS

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REFERENCES

Alben, K., and E. Sphirt. 1983. Distribution profits of chloroform, weak organic acids and PCBs on granular activated carbon columns from Waterford, New York. Environmental Science and Technology, 17(4): 187-192.

Alben, K., E. Sphirt, and N. Perrins. 1983. Experimental profiles of organic and inorganic substances adsorbed on fixed beds of granular activated carbon, in Treatment of Water by Granular Activated Carbon. American Chemical Society, Washington, 599 pp.

Andrews, R.C. 1986. Distribution of Trihalomethanes and Total Organic Carbon in Pilot Scale GAC Columns, M.Sc. thesis, University of Alberta, Edmonton.

Andrews, R.C., L. Gammie, and P.M. Huck. 1983. Distribution of volatile organics in pilot scale GAC columns. Presented at the 66 Annual Conference, Chemical Institute of Canada, Calgary, Alberta, June.

Cairo, P.R., J. McElhaney, and I.H. Suffet. 1979. Pilot plant testing of activated carbon adsorption systems. J. AWWA., 71(11): 660-673.

CH2M Hill. 1980. Water treatment systems improvement and expansion studies for the Buffalo Pound Water Administration Board, Calgary, Alberta, December.

Crittenden, J.C. and D.W. Hand. 1983. Design considerations for GAC treatment of synthetic chemicals and TOC. Proceedings of AWWA Conference, Las Vegas, Nevada.

Fochtman, E.G. and R.A. Dobbs. 1980. Adsorption of carcinogenic compounds by activated carbon, in Activated Carbon Adsorption of Organics from the Aqueous Phase, Vol. 1. Ann Arbor Science Publishers Inc., Ann Arbor, Michigan. 508 pp.

Forsyth, D.E., L. Gammie, X. Bing-song, and P.M. Huck. 1982. Pilot scale activated carbon studies at Buffalo Pound. Proceedings of the symposium on wastes and prairie water quality. Saskatchewan Water Studies Institute, Saskatoon, May.

Kreff, P., A. Lang, M. Kavanaugh, and R. Trussell. 1981. Notes and comments: Leaching of organics from a PVC-polyethylene-plexiglass pilot plant. J. AWWA., 73(10): 558-560.

Maloney, S.W., K. Bancroft, and I.H. Suffet. 1983. Comparison of adsorptive and biological total organic carbon removal by granular activated carbon in potable water, in Treatment of Water by Granular Activated Carbon. American Chemical Society, Washington, 599 pp.

McGuire, M.J., T.S. Tanaka, and M.K. Davis. 1983. Experimental error estimates associated with pilot activated-carbon investigations of trace organic removals, in Treatment of Water by Granular Activated Carbon. American Chemical Society, Washington, 599 pp.

Mullins, R.L., J.S. Zogorski, S.A. Hubbs, and G.D. Allgeier. 1980. The effectiveness of several brands of granular activated carbon for the removal of trihalomethanes from drinking water, in Activated Carbon Adsorption of Organics from the Aqueous Phase, Vol. 1. Ann Arbor Science Publishers, Inc., Ann Arbor, Michigan, 508 pp.

Neely, D.W., and E.G. Isacoff. 1982. Carbonaceous adsorbents for the treatment of ground and surface waters. Marcel Dekker Inc., New York, 228 pp.

Norusis, M.J. 1983. SPSS^X Introductory statistics guide. New York, McGraw-Hill Book Company, 276 pp.

Rosene, M.R., R.T. Deithorn, J.R. Lutchko, and N.J. Wagner. 1980. High-pressure technique for rapid screening of activated carbons for use in potable water, in Activated Carbon Adsorption of Organics from the Aqueous Phase, Vol. 1. Ann Arbor Science Publishers, Inc., Ann Arbor, Michigan, 508 pp.

USEPA. 1980. Removing potential organic carcinogens and precursors in drinking water. U.S. Environmental Protection Agency Rep. No. EPA-600/2-80-130a, 367 pp.

Weber, W.J. ed. 1972. Physiochemical processes for water quality control. New York, N.Y., Wiley-Interscience, 640 pp.

Weber, W.J., T.C. Voice, and A. Jodellah. 1983. Adsorption of humic substances: the effects of heterogenicity and system characteristics. J. AWWA. 75(12):612-619.

Wood, P.R., and J. DeMarco. 1979. Treatment of groundwater with granular activated carbon. J. AWWA., 72(11):674-682.

Yohe, T.L., I.H. Suffet, and P.R. Cairo. 1981. Specific organic removals by granular activated pilot contractors. J. AWWA., 73(8): 402-410.

Zogorski, J.S., and S.D. Faust. 1978. Operational parameters for optimal removal of phenolic compounds from polluted waters by columns of activated carbon, in Carbon Adsorption Handbook, Ann Arbor Science Publishers, Inc., Ann Arbor, Michigan, 1054 pp.

WATER QUALITY AS AFFECTED BY PRAIRIE GEOLOGY

by

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INTRODUCTION

The true prairie region of the Prairie Provinces and some parts of the northern states of the U.S.A. share a common geological heritage. Vast deposits laid down as a result of a number of glaciations form the surface deposits. The general slope of the land is from the west to the east and from the south to the north.

As the continental ice sheets retreated towards the north, large channels were formed which carried the melt-water from the glaciers, usually southward and eastward, to points of storage or discharge.

The landforms of the plains as they exist today are the result of erosion and deposition by ice and melt-water associated with the retreat of the last glacial age. The last continental glacier disappeared from the prairies about 10,000 to 17,000 years ago.

LANDFORMS

Uplands

With the exception of the west block of the Cypress Hills, most uplands in the prairies are composed of several layers of drift deposited by the continental glaciers as they advanced and retreated, stalled and again retreated, time after time.

In Saskatchewan, predominant upland features include the Missouri Coteau, the Allan Hills, the Strawberry Hills, the Pasque Hills, Moose Mountain, etc.

Rivers

The major river system of the Prairies is that of the North and South Saskatchewan and its tributaries. (Figure 1) The major branches arise in the Rocky Mountains. The North branch flows through Edmonton and the Battlefords, then Southeast toward Saskatoon, but turns northeast to pass through Prince Albert. The South branch flows through Calgary as the Bow River, then easterly through Medicine Hat. It turns north-easterly just northwest of Swift Current, (where the valley is flooded as Lake Diefenbaker behind Gardiner Dam), then northward with easterly settings through Saskatoon to the confluence with the North branch some 100 km downstream of Prince Alberta.

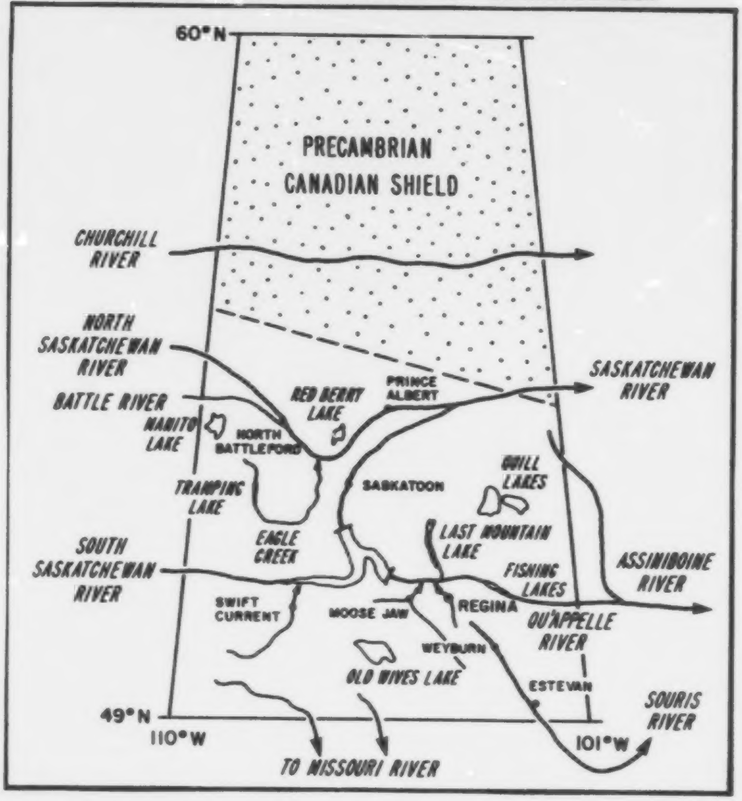
The main tributary joining the North Saskatchewan in Saskatchewan is the Battle River which arises in Central Alberta and enters the North Saskatchewan just south of the Battlefords.

A number of tributaries combine with the South Saskatchewan in Alberta;

however, the largest tributary in Saskatchewan is the Swift Current Creek which arises in the Cypress Hills and enters the South Saskatchewan northeast of Swift Current.

Figure 1

Surface Water Resources of Saskatchewan


60°N
PRECAMBRIAN CANADIAN SHIELD
CHURCHILL RIVER
NORTH SASKATCHEWAN RIVER
BATTLE RIVER
MANITO LAKE
NORTH BATTLEFORD
RED BERRY LAKE
PRINCE ALBERT
SASKATCHEWAN RIVER
TRAMPING LAKE
SASKATOON
QUILL LAKES
EAGLE CREEK
LAST MOUNTAIN LAKE
SOUTH SASKATCHEWAN RIVER
FISHING LAKES
ASSINIBOINE RIVER
SWIFT CURRENT
MOOSE JAW
REGINA
QU'APPELLE RIVER
WEYBURN
OLD WIVES LAKE
ESTEVAN
SOURIS RIVER
49°N
110°W
101°W
TO MISSOURI RIVER

Minor streams which are tributary to the eastward flowing Missouri River arise in the extreme southerly portion of Alberta and Saskatchewan and flow southward into the United States.

In order to retain Lake Diefenbaker in the Valley of the South Saskatchewan, a dam had to be constructed at the divide of the South Saskatchewan drainage basin and the Qu'Appelle drainage basin. Water is pumped over this dam from Lake Diefenbaker and forms an instant head-water of the Qu'Appelle River. Between 2 and 6 cubic metres per second have been diverted continuously to "flush out" Buffalo Pound Lake and maintain the level of the Fishing Lakes.

The Qu'Appelle River meanders in a proglacial valley [melt-water channel] nearly due east. It receives tributary flow from Thunder Creek, and the Moose Jaw River which enters the Qu'Appelle just below the Dam forming Buffalo Pound Lake. Just above Lumsden, the Qu'Appelle receives the flow from Wascana Creek with its wastewater contribution from the city of Regina, and at Craven the overflow from Last Mountain Lake enters the Qu'Appelle. Tributaries to this very large body of water drain the Great Salt Desert south of the Allan Hills and west of the Touchwood Hills. The Arm River which drains the west side of the Allan Hills enters the Lake just before its outlet. A few small creeks and rivers enter the Qu'Appelle prior to its crossing the Manitoba boundary, where soon after it joins the Assiniboine River draining the area South of the Porcupine Hills. The Assiniboine proceeds East to join the Red River at Winnipeg.

The Souris River rises 40 or so kilometres North and West of Weyburn and flows by Estevan and leaves the province at the extreme southeast corner where it becomes the Mouse River during its flow through North Dakota before it again crosses the 49th parallel to once again become the Souris on its passage to the confluence with the Assiniboine River near Brandon.

Saline Lakes

Much of the remaining part of the Province of Saskatchewan is covered with dead drainage areas where the stream flows enter a body of water which has no outlet. Runoff supplemented by groundwater discharge followed by evaporation has left these as Saline Lakes. Typical examples are Manito Lake south of Lloydminster, Redberry Lake north of Saskatoon, and the Quill Lakes 100 km east of Saskatoon. These now find limited use only as recreational water bodies, although commercial fishing was in operation in the Quill Lakes within living memory.

Literally hundreds of "Alkali" lakes are located in the Prairie provinces, varying in size from the long narrow lakes such as Blackstrap occupying a proglacial melt-water channel to closed basins such as Old Wives Lake covering up to 300 square kilometers. The broad Alkali lakes are customarily found on the flats below the upland areas, while those confined in the melt-water channels are randomly located and randomly oriented. The high mineral salt content of the water is the result of groundwater flow into the depressions. Groundwater dissolves highly soluble salts such as Sodium Sulfate (Na_2SO_4 or Alkali) as it flows through the drift deposits.

A number of lakes support Sodium Sulfate recovery industries which have been in operation for 50 years or more. The recovery processes are usually based on the difference in solubility of Na_2SO_4 in water at warm and at cold temperatures.

Lake water (brine) is directed into dyked holding basins during the summer when the salt content is high. Under water temperatures the brine becomes supersaturated and crystals of Na_2SO_4 precipitate. The mother brine is discharged leaving the salt in the ponds as raw product for the next years harvest and refining.

The definitive work on the Alkali lakes of Western Canada details a number of lakes which were found to contain from small to extremely large deposits of solid Sodium Sulfate (Cole L.H. 1926).

WATER SUPPLY

The supply of good quality water in the Prairie Provinces can be described as being sufficient, adequate, or potentially dangerously low when the demands of domestic supply,industrial supply, recreational supply and agricultural supply are considered. In the total picture, domestic and recreational demands are not consumptive demands. Industrial demands may have a consumptive component; however, irrigational demands is mainly consumptive, under the present practices of irrigation. Whatever portion of the flow is not consumed is greatly reduced in quality, usually as a result of an increase in the mineral salt content load it carries.

The amount of irrigated land on the prairies and particularly in Saskatchewan has increased greatly over the last 10 years. More importantly, the practices of applying irrigation water have changed dramatically. The flood irrigation and ridge and furrow irrigation methods which were so labour intensive have been superceded by spray irrigation using large machines. Centre Pivot machines capable of irrigating the better portion of a section of uneven ground are in common use and their numbers are increasing. This means a startling increased demand for cheap water; a demand that is so large that it can only be met by utilizing water from the permanently flowing river systems in the province. This essentially means the Saskatchewan River system and more the specifically the South Saskatchewan River.

Concerns have already been expressed regarding the ability of the South Saskatchewan River to meet the short and long term demands which will be placed on it, to supply domestic and irrigation water to the majority of, and the driest portion of, the agricultural area of the province.

Because of an Equitable Apportionment agreement between the Prairie provinces, Saskatchewan is entitled to 50 percent of the historical flows in the rivers flowing from Alberta, and must pass 50 percent of the historical flows to Manitoba.

Within the province, the ability to manipulate the flows of the Saskatchewan River within its basin are provincial rights provided the Inter-provincial rights are not violated.

WATER DIVERSION SCHEMES

One scheme receiving support and consideration is the diversion of water from the North Saskatchewan River near North Battleford south to the South Saskatchewan River in the Eston area, upstream of the main portion of Lake Diefenbaker. This scheme envisions pumped and gravity flow of water and storage of the water in the natural depressions in the area. Because of its approximately 40 kilometer length and its predominantly North-South orientation Tramping Lake would appear to be a pivotal geographic feature in this scheme to utilize the large valley of Eagle Creek. About 80,000 square kilometers of the driest area of the province would be opened to irrigation if the diversion were to take place.

Two diversion schemes from the South Saskatchewan already exist. Both are bound to get large; but more importantly, both result in a major deterioration of the quality of the diverted water.

1. The Qu'Appelle Valley Diversion

The first existing scheme is the aforementioned diversion from Lake Diefenbaker to the Qu'Appelle River. The flow of 2 to 6 m^3/sec is insufficient to flush out Buffalo Pound Lake and thereby improve the quality of the raw water supply of the cities of Regina and Moose Jaw. This scheme has been in place for about 15 years with no appreciable improvement to the water supply for the two cities.

The concept that this small amount of diverted flow could stabilize the level of the Fishing Lakes and have a beneficial impact on the water quality of those lake denies the origin of the deep lakes in a melt-water channel which throughout the majority of its length is filled with over 30 metres of alluvium.

An acceptable theory of the origin of the lakes was presented by Christiansen et al (1981).

Figure 2 is an aerial photograph of the Fishing Lakes which clearly shows how the original Fishing Lake was divided into five lakes by the alluvial fans (deltas) formed at the outlets of water courses eroding deeply into the surrounding plain. The major fan closing the valley in the centre of the Figure is the location of the present town of Fort Qu'Appelle, which started as the first administrative centre in the valley west of Winnipeg during the time of the opening of the West in the middle 1800's.

The form of the valley containing the lakes is typical of the valley formed by proglacial streams. The valley is deep, sharp banked with numerous bank erosion scars and is of considerable length.

Figure 3 shows the stratigraphy and groundwater flow system of a cross-section of the Qu'Appelle Valley between the lower two lakes in Figure 2.

The Qu'Appelle Valley channel carried melt-water from the front of the receding glacier towards the east. According to Christiansen et al (1980), the valley eroded deeply and finally intersected the buried Hatfield Valley Aquifer. The very high velocity gradients associated with massive upflows of water from the aquifer carried the alluvial material out of the immediate area leaving a large deep lake where the Qu'Appelle Valley Channel intersected the Hatfield Valley Aquifer. The effect of the vertical and horizontal currents was overcome by delta formations which seperated the single proglacial Fishing Lake into the present Fishing Lakes.

There is still a major contribution of groundwater into the Fishing Lakes which dictates in part the Lake levels and the chemical constituents of the water in the lakes.

"...the Hatfield Valley aquifer drains into the Qu'Appelle Valley near Fort Qu'Appelle where major discharge appears to be concentrated into the Fishing Lakes, in particular Mission Lake... The water level in the Hatfield Valley aquifer declines toward the Qu'Appelle Valley, which intersects the aquifer and acts as an enormous drain." (Schreiner B.T. and Maathius, H. 1982). The average quality of the water in the Hatfield Valley aquifer is given in the same report. The concentration of the Total Dissolved Solids on the average is 2260 mg/l making the water marginal to unacceptable as a drinking water source for municipal supply. The combination of a high Apparent Sodium Adsorption Ratio and the high mineral salt concentration makes the water unfit for irrigation purposes."

The mineral salt content of the Hatfield Valley Aquifer is thus from 8 to 9 times that of water in the South Saskatchewan River. Any storage of South Saskatchewan River water in the Fishing Lakes would result in a marked deterioration in proportion to the relative contributions from the river and from the Hatfield Valley aquifer.

Figure 2

Aerial Photograph of the Fishing Lakes

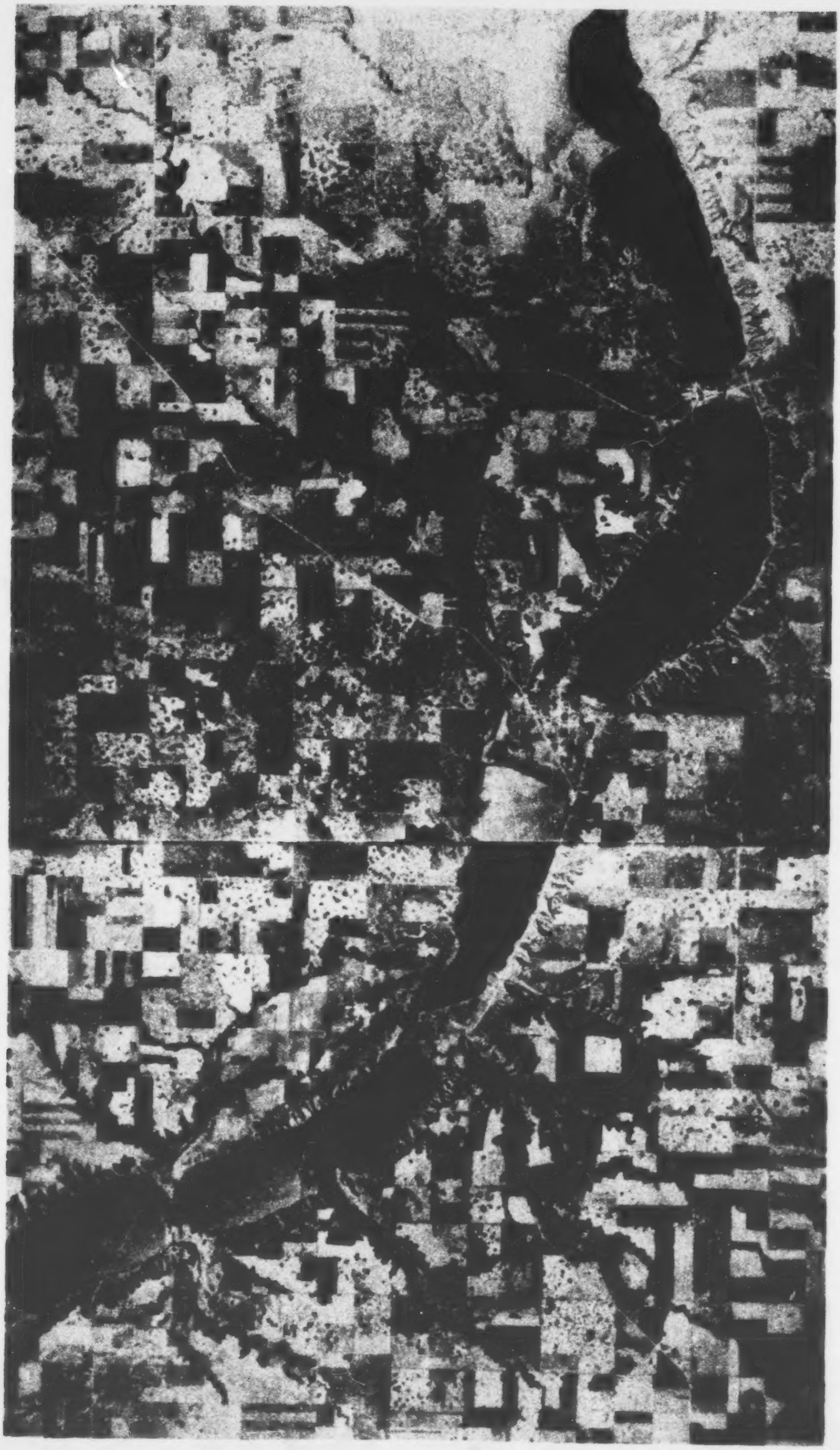
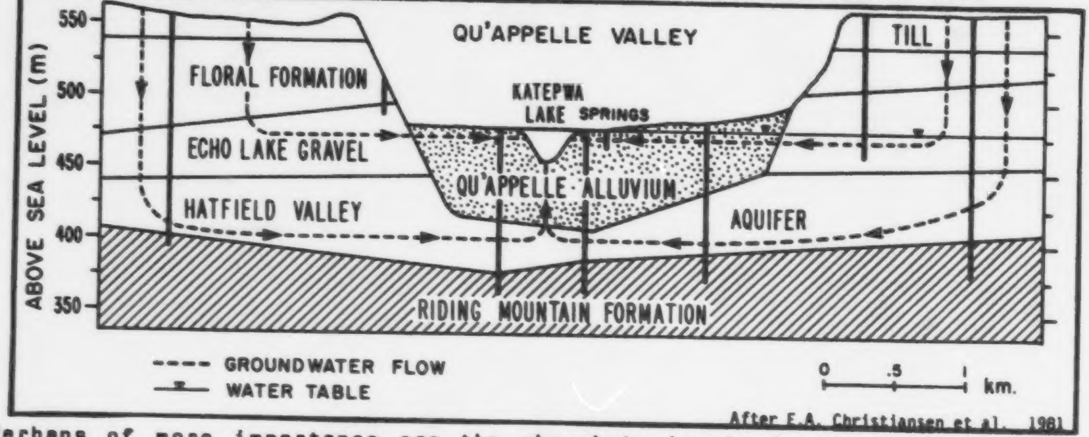


Figure 3

Cross-Section of Qu'Appelle Valley at Katepwa Provincial Park


550
500
450
400
350
ABOVE SEA LEVEL (m)
QU'APPELLE VALLEY
TILL
FLORAL FORMATION
KATEPWA LAKE
SPRINGS
ECHO LAKE GRAVEL
QU'APPELLE ALLUVIUM
HATFIELD VALLEY
AQUIFER
RIDING MOUNTAIN FORMATION
GROUNDWATER FLOW
WATER TABLE
0
.5
1
km.

After E.A. Christiansen et al. 1981

Perhaps of more importance are the phosphate levels in the Hatfield aquifer water. According to the analyses published (Schreiner B.J. and Maathius, H. 1982), the average PO_4 content of the Hatfield water 0.4 mg/l with a standard deviation of 0.4 mg/l. This average value is about 30 times the minimum critical amount of PO_4 required to support blooms of algae.

A modified Vollenweiter diagram (Figure 4) containing the information on total estimated phosphorous inputs to the Fishing Lakes indicates they receive at least 10 times the phosphorous load of Lakes Erie and Ontario (Barcia, J. 1980). To bring the Fishing Lakes to a Mesotrophic state their phosphorous load would have to be reduced by up to two orders of magnitude. Such a reduction does not seem feasible in an area rich in natural and applied fertilizers which enter the lakes from point and non-point sources. Indicators are that the "Green Lakes" will always be a major water quality sink.

2. Southeast Saskatoon Water Supply System

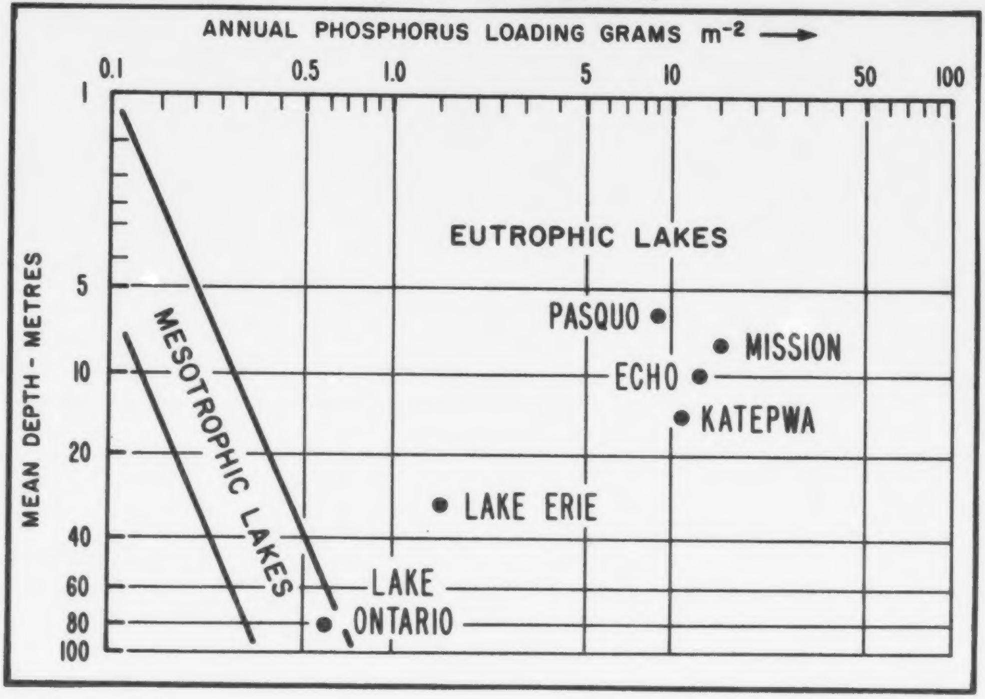
The existing South-East Saskatoon Water Supply System (SSWSS) was constructed to supply water from Lake Diefenbaker (South Saskatchewan River) to be used for industrial, domestic, agricultural and recreational purposes.

The water is pumped from the lake and conveyed by gravity canals to five storage reservoirs in series. Provision is made for water which is allowed to flow through the fifth storage reservoir to be pumped so as to flow into a sixth storage reservoir.

The soil through which the distribution canals are constructed for the first half of the system, and the first two reservoirs are "Passive" in that they do not contribute dissolved minerals to the water as it passes through them. (Davis and Sauer, 1983).

Figure 4

Eutrophication - Phosphorus Loading Relationship
[After Vollenweiter]



The water, as it is pumped from Lake Diefenbaker, is of excellent quality, based on Western Canadian drinking water quality standards. It is a Calcium-Bicarbonate water with typically 260 to 290 mg/l of Total Dissolved Solids [TDS] present. The water remains at this level until it is discharged into Blackstrap reservoir.

Immediately the good quality water enters this reservoir, its quality undergoes a deterioration as far as TDS is concerned and the water changes to a Sodium Sulfate type. Deterioration progresses as the water moves to the reservoir outlet. Blackstrap is now recognized as an "Active" reservoir.

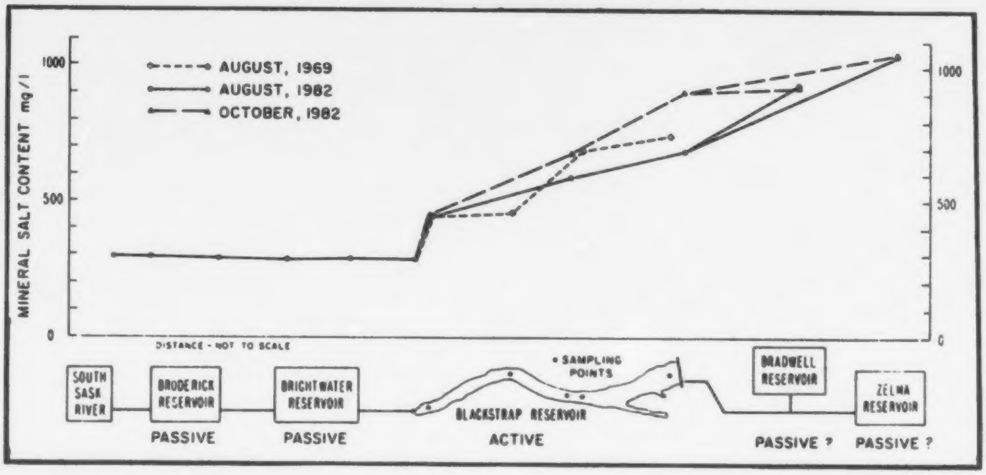
Figure 5 shows the water quality deterioration from inlet to outlet of Blackstrap reservoir and beyond. Table 1 details the water quality deterioration which took place as the water moved from inlet to outlet. The values given for August 1982 reflect the dilution effect of an inlet flow. The October values [long dashed line] reflect that no flow was entering the reservoir and the TDS content increased significantly to greater than 900 mg/l. This is greater than a three-fold increase from the TDS value of the inlet water.

As benchmark values the TDS determined in 1969, two years after the reservoir was first placed in operation, are included in Figure 5. The maximum 1969 value falls between the maximum values determined in 1982.

Investigation of aerial photographs taken in 1955, some 10 years before the reservoir was constructed, showed the north end of the Blackstrap to be an area of alkali accumulation. See Figure 6. Alkali (Sodium Sulfate) deposits have long been recognized as characteristic evidence of a groundwater discharge area.

Figure 5

Mineral Salt Content of Water



Blackstrap valley shows the same proglacial valley characteristics given relative to the Qu'Appelle Valley.

Figure 7 shows how the Blackstrap Reservoir has flooded the Alkali area of the valley and how the reservoir has been kept in the main valley by a dam on a small abandoned proglacial valley.

Figures 6 and 7 may be matched based on the location of the causeway constructed normal to the valley at the mouth of the major water course draining into the Blackstrap Valley from the east.

Christiansen (1975) described the stratigraphy of a cross-section across the Blackstrap Valley at the Causeway. See Figure 8. It is evident that the Blackstrap Reservoir is built on the alluvium some 150 feet deep, filling a melt-water channel. The Blackstrap Valley is itself cut into a much larger and deeper (800 feet) valley which penetrates the Judith River Formation.

The Judith River Formation is artesian and supplies the necessary hydraulic head to make Blackstrap a continuing groundwater discharge area. Nothing is known about the quantity of groundwater discharging into the Blackstrap Reservoir and very little is know about the quality of the water in the Judith River formation at that location. The quality of groundwater deteriorates however, as it flows upward through drift deposits, typically accumulating Sodium and Sulfates.

The water which is discharged from Blackstrap Reservoir runs in canals north and east through "Alkali" country. A side reservoir is located at Bradwell, but the main flow continues east to Zelma reservoir. This reservoir is similar in nature to Blackstrap Reservoir in that it is a dammed-off portion of an "Alkali" depression. No stratigraphic evidence has been documented that would shed light on the groundwater discharge mechanism at Zelma. From Zelma

Figure 6

Aerial Photograph of Blackstrap Valley - 1955

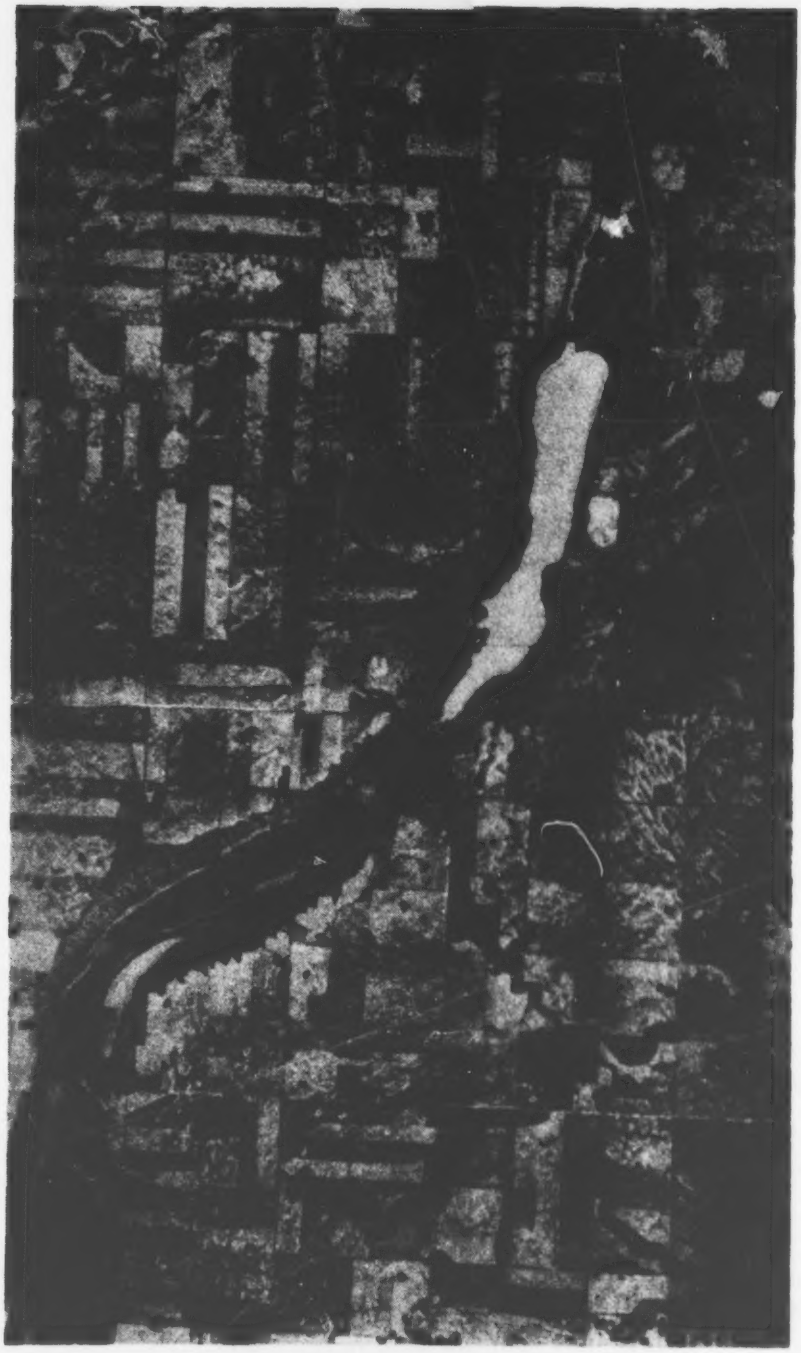
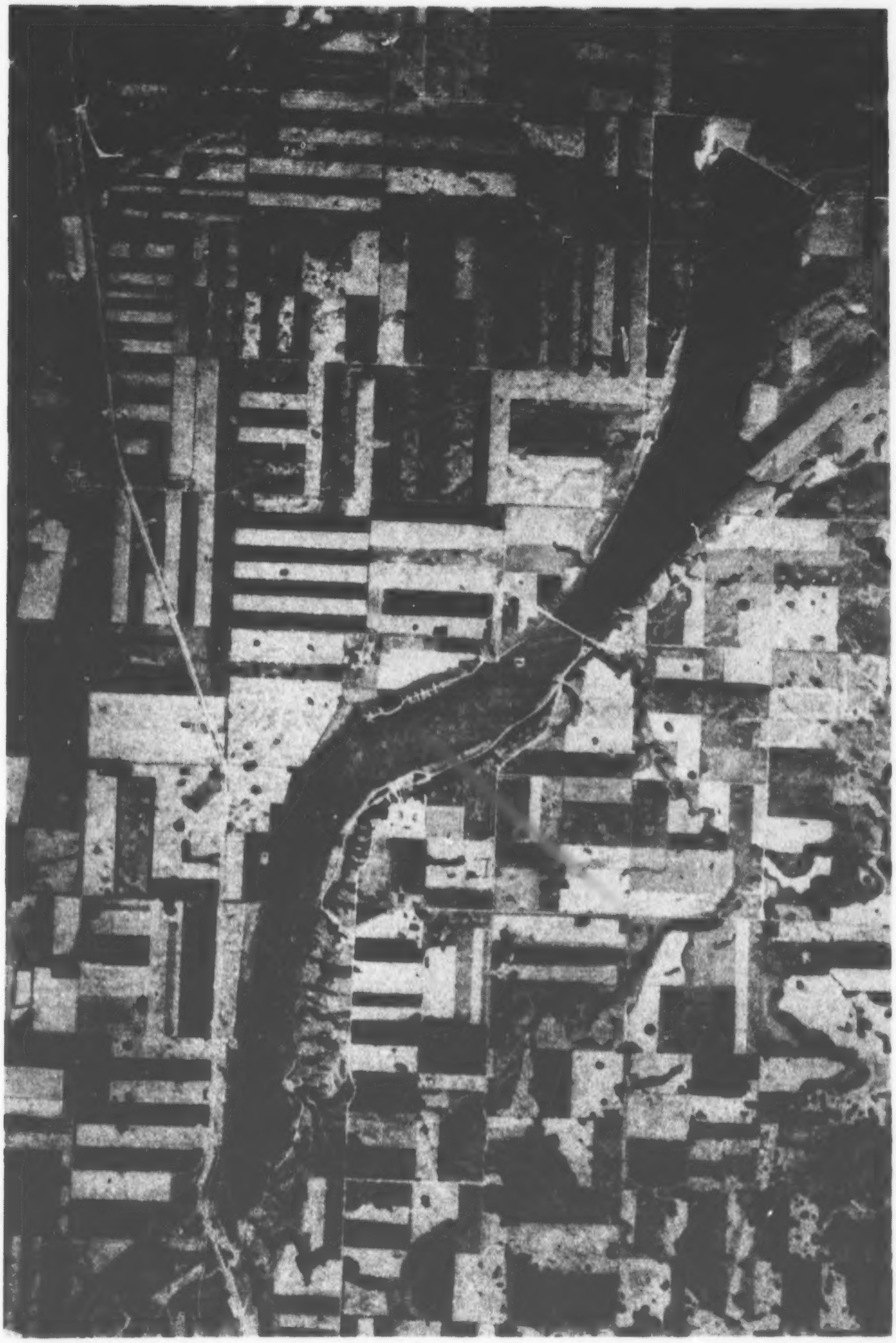


Figure 7

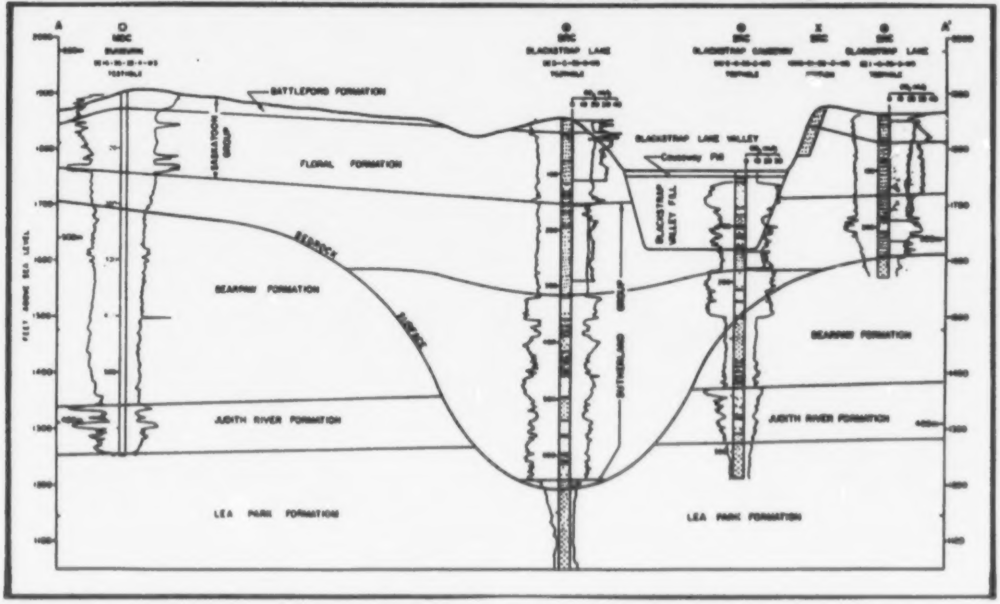
Aerial Photograph of Blackstrap Reservoir - 1972



Reservoir water can be discharged into Little Manitou Lake or it can be pumped up onto the north side of the lake valley and run by gravity into Dellwood Reservoir which can be classified as a Passive reservoir. However, the previous abuse to which the water has been subjected makes the water in Dellwood Reservoir one of very low quality and very difficult to treat in a small conventional water treatment plant.

Figure 8

Cross-Section across Blackstrap Lake Valley


A
A'
BATTLEFORD FORMATION
SASKATOON GROUP
FLORAL FORMATION
BLACKSTRAP LAKE VALLEY
Causeway Fill
BLACKSTRAP VALLEY FILL
BEDROCK SURFACE
BEARPAW FORMATION
SUTHERLAND GROUP
JUDITH RIVER FORMATION
LEA PARK FORMATION
BLACKSTRAP LAKE
TESTHOLE
FEET ABOVE SEA LEVEL
1900
1800
1700
1600
1500
1400
1300
1200
1100

3. North Saskatchewan River to South Saskatchewan River Diversion

Mention was made previously of a plan to divert water from the North Saskatchewan River by pumping the water to the south and distributing it in the dry west central portion of the province. Tramping Lake has been identified as a possible storage and transport reservoir because of its large size and long North-South orientation. Recent investigative work on Tramping Lake, facilitated by the construction of a highway causeway across the lake has shed light on the stratigraphy of the area. Figure 9 shows Tramping Lake as a typical proglacial water course.

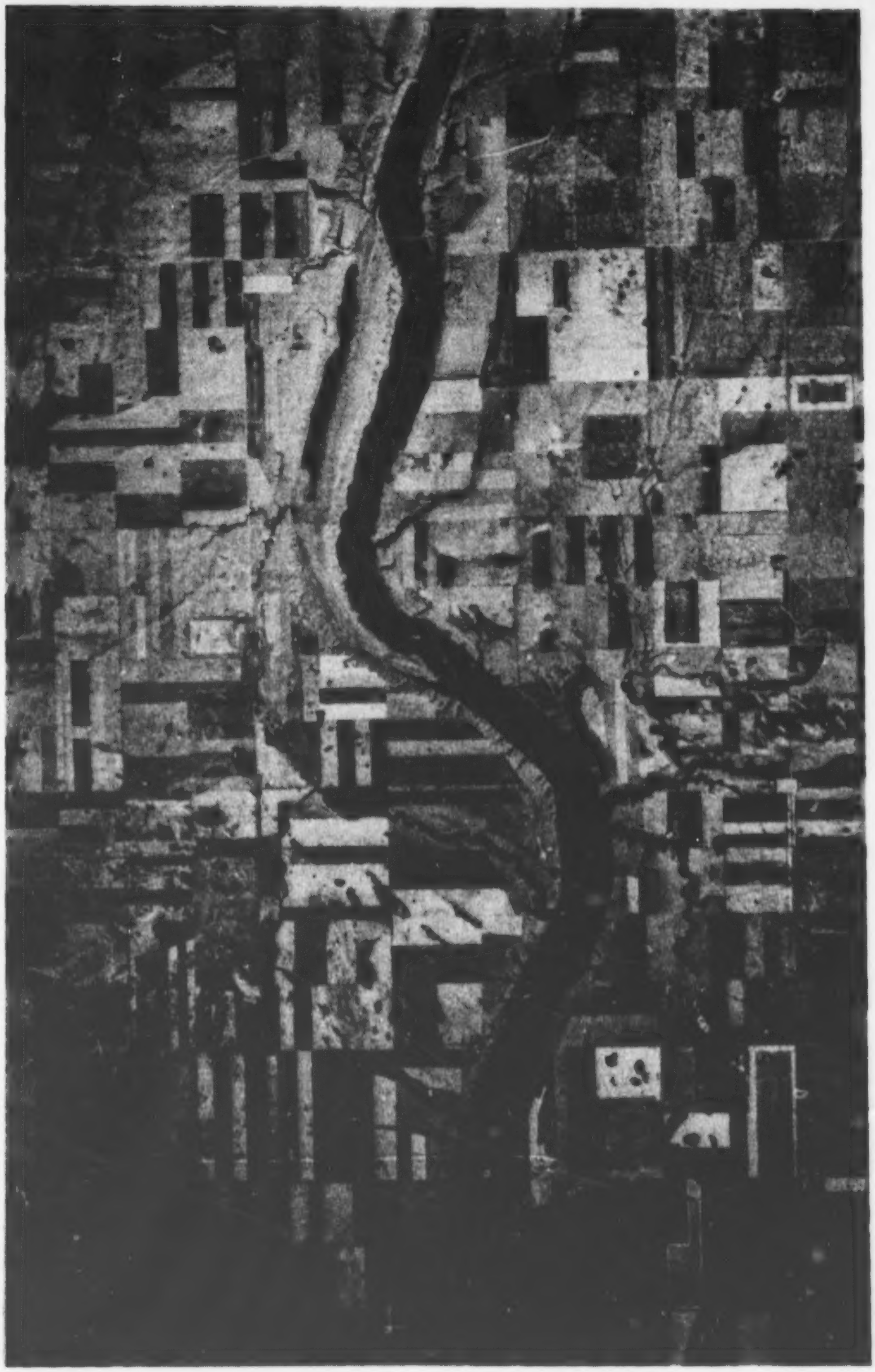
Figure 10 shows the stratigraphy of Tramping Lake at the Causeway. Tramping Lake Valley was eroded through the drift and some 28 metres into the Judith River Formation. The majority of the bottom of the valley has been filled with alluvium. The water level in the Lake is just below the top of the Judith River Formation.

A series of piezometers set at different levels in the alluvium and in the Judith River Formation indicate an increasing hydraulic head with depth which provides a potential for an upward flow of water into the lake bottom.

The quality of the water obtained from the piezometers deteriorates rapidly as it approaches the lake bottom. The TDS in the Judith River Formation is 1160 mg/l 88 metres below the lake surface. The TDS of water obtained from a

Figure 9

Aerial Photograph of Tramping Lake

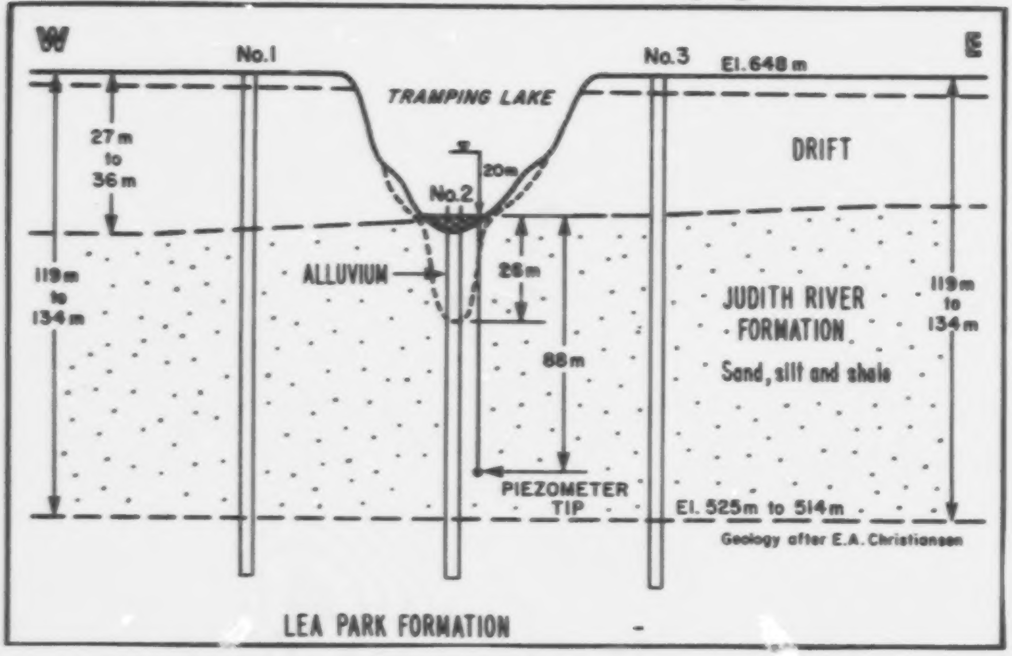


piezometer was 3920 mg/l 23 metres below the lake surface, and the lake water contained 12,380 mg/l of TDS.

Based on this latter figure, there are approximately one half million tons of mineral salts in the water in Tramping Lake. This, if bled over a one year period, is sufficient to double the mineral salt level of the North Saskatchewan River during that period.

Figure 10

Stratigraphic Cross Section of Tramping Lake


W
No.1
No.3
El.648 m
E
TRAMPING LAKE
DRIFT
27m to 36m
20m
No.2
ALLUVIUM
26m
119m to 134m
JUDITH RIVER FORMATION
Sand, silt and shale
119m to 134m
88m
PIEZOMETER TIP
El. 525m to 514m
LEA PARK FORMATION

Geology after E.A. Christiansen

Core samples of the alluvium from the lake bottom indicate a high concentration of crystals of salt in the bottom muds. Figure 11 shows the variation of the concentration of Sodium ions and Sulfate ions with depth. Making certain simplifying assumptions where firm data are not yet available, and recognizing that the concentration of the salts in the soil and in the lake water must be equal at the soil-water interface, then there are by calculation about 75 million tonnes of mineral salts in the muds of the lake bottom which is maintained by the upward flow of groundwater.

This vast reservoir of mineral in the lake bottom precludes the use of Tramping Lake as a transportation and storage reservoir as any fresh water entering the lake would be degraded in quality in proportion to (1) the quantity of ground water discharge into the lake bottom and (2) the quality of that discharge.

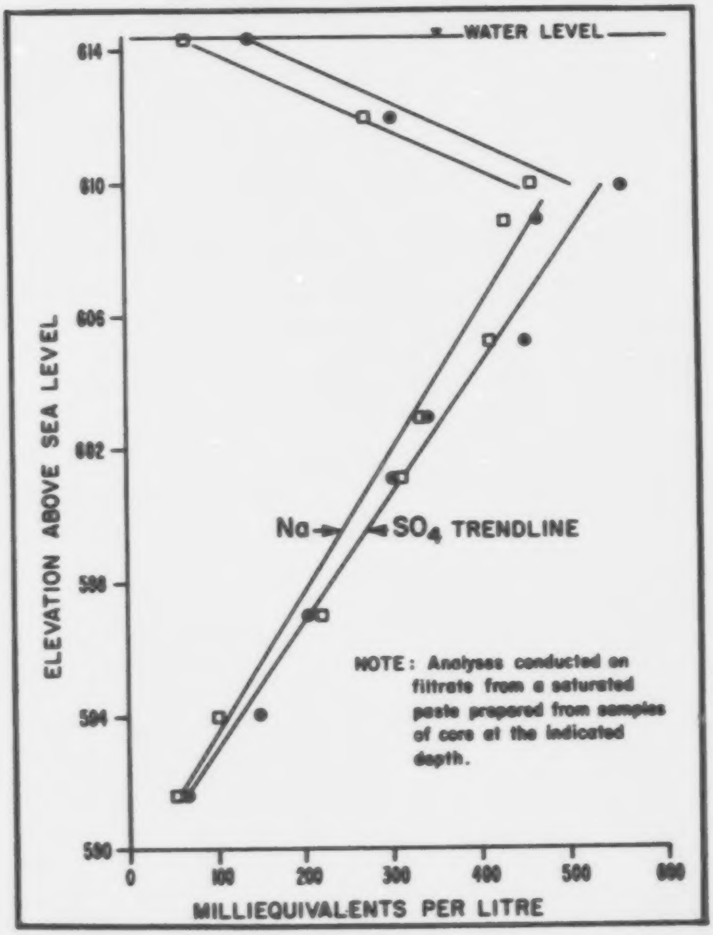
SUMMARY

An actual case of water quality deterioration resulting from using groundwater discharge areas as a storage and transportation reservoir has been presented. This involves the Blackstrap reservoir.

Two other cases have been identified which would result in water quality deterioration if the areas are used as storage and transportation reservoirs. The involve the Fishing Lakes and Tramping Lake.

Figure 11

Depth-Concentration relationship of Major Ions in Tramping Lake Alluvium



NOTE: Analyses conducted on filtrate from a saturated paste prepared from samples of core at the indicated depth.

CONCLUSIONS

1. The demand for the movement of fresh water throughout Western Canada will be increased with increasing population and increasing change of irrigation practices.

2. Many water transportation routes utilizing natural pro-glacial depressions and lakes are not open to use because they will cause water quality deterioration. This is because areas of low elevation are potential groundwater discharge areas.

3. The input of minerals into reservoir water originates as groundwater. The flow of groundwater is controlled by the sequence and topographic

configuration of layers of deep-deposits, therefore a thorough understanding of geology, particularly stratigraphy of the drift is a required input to the planning of any water transportation schemes.

4. Much more investigation is required in order to establish criteria for transportation routes which are satisfactory or unsatisfactory for the movement of good quality water.

5. Much more investigation is required to quantify the following parameters in groundwater discharge areas:

   a) the hydraulic head
   b) the actual flow rate
   c) the quality of the water at discharge

ACKNOWLEDGEMENTS

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REFERENCES

Barica, J., 1980. "Some Biological Characteristics of Plains Aquatic Ecosystems and their Effect on Water Quality" Prairie Surface Waters: Problems and Solutions. Canadian Plains Research Centre, University of Regina, Regina, Saskatchewan, pp. 37-48.

Christiansen, E.A., Acton, D.F., Long, R.J., Meneley, W.A., and Sauer, E.K. 1981. "Fort Qu'Appelle Geolog - The Valleys - Past and Present". Saskatchewan Culture and Youth, Regina, Saskatchewan". Geological Association of Canada Guidebook, 81 p.

Christiansen, E.A. 1973. "Quarternary Geology and its Application to Engineering Practice in the Saskatoon - Regina - Watrous Area, Saskatchewan". Geological Association of Canada Guidebook, 81 p.

Cole, L.H. 1926. "Sodium Sulfate of Western Canada" Canada Department of Mines, Ottawa, Canada. Mines Branch Report 646.

Davis, E. and Sauer E.K., 1983. "Characterization of Reservoirs in Southern Saskatchewan: Active or Passive". Proceedings Prairie Hydrology Technical Session, Canadian Water Resources Association, Published by Water Studies, Institute, University of Saskatchewan.

Sawyer, C.L. and McCarty, P.L., 1978. "Chemistry for Environmental Engineering". McGraw-Hill Book Company, New York, N.Y.

Schreiner, B.T. and Maathius, H., 1982. "Hatfield Valley Aquifer System in the Melville Region, Saskatchewan" Volume 1 Saskatchewan Groundwater Investigations, Saskatchewan Research Council Publication No. G-743-3-B-82.

PERFORMANCE OF THE INTEGRATED CONTACT CLARIFIER/FILTRATION PROCESS

by

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INTRODUCTION

Enhanced water treatment technologies are continually sought out in the industry to provide more effective treatment at reduced costs. The industry has seen several advances over the years, but the most significant advanced technologies that have survived trends and gained acceptance are shallow depth sedimentation, high rate filtration and coagulation control.

A technology now gaining broader acceptance is upflow buoyant media contact clarification, also known as "Adsorption Clarification." This recently developed technology combines the function of flocculation and clarification and is typically coupled with a Mixed Media Filter for complete treatment. Packaging this system with a microprocessor coagulation controller creates one of the newest enhanced treatment technologies; Trident® Water Systems.

NEW ADSORPTION CLARIFICATION TECHNOLOGY

In 1982 the Adsorption Clarifier^{tm} was introduced into the market as part of an upflow/downflow, two stage process. The clarifier is an upflow device that uses buoyant plastic media. The concept of a fixed-medium clarification device is a recent adaptation of several older concepts.

This process effectively replaces the flocculation and settling functions of conventional water treatment systems. When passing through a bed of media, chemically coagulated water is subjected to the combined effects of contact flocculation and clarification.

Flocculation

As water passes through the clarifier bed, mixing occurs as a result of the energy expended in the development of headloss through the angular, randomly oriented media. The natural result of this mixing is flocculation of particles within the bed. This phenomenon has been studied in the past [2,3,4] and has been termed contact flocculation or particle flocculation. The process is enhanced by contact with previously deposited solids. The contact flocculation process thus borrows some of the theoretical advantages of solids contact clarifiers, but is not limited by the vagerious settling properties of the solids. Careful attention to design or operation is not required in order to maintain a stable solids inventory within the system.

A common parameter in flocculation is the dimensionless product of the mixing intensity parameter and detention time, Gt. A calculation of the mixing intensity parameter (G) in a clean bed with 6 inches headloss at 10 gpm/sq. ft. gives a value of about 110/sec. This increases to G values approaching 300/second in a bed at 4 feet headloss. Calculated values of Gt are in the range of 1.0×10^4 to 3.0×10^4, depending on headloss and flux rate. These values compare well with data presented by Trewsek (5) which showed optimum Gt values of 1.2×10^4 to 4.2×10^4 for flocculation systems preceding direct filtration. Flocculation within the Adsorption Clarifier should be superior to conventional bulk flocculation due to the presence of high concentrations of attached solids.

Modelling of the kinetics of the flocculation process within an Adsorption Clarifier is elusive. Particle counts have been performed on influent and effluent streams, which reveal reductions in the number of particles of all sizes, with only a slight shift in the relative distribution. As pointed out by both Saleh and Cleasby (3) and by Ives (4), the fact that solids deposition is occurring within the media, as well as probable particle detachment, obscures the results. The observations of these authors have been confirmed; when a run is allowed to proceed until a breakthrough occurs, the solids seen leaving the media are large and well flocculated. Particle count analyses have confirmed that escaping particles increase in size throughout the length of clarifier run. These larger particles are well flocculated and conditioned for easy removal within the second stage Mixed Media filter bed.

Clarification

Clarification, or turbidity removal, is the primary function of the Adsorption Clarifier. It is accomplished through straining and adsorption of the coagulated particles onto the surfaces of the media and to other previously attached solids. Turbidity removals through the clarifier are typically in the range of 70 to 95 percent, which compare favourably to results obtained with conventional clarification systems.

In the clarification/filtration system, the focus of turbidity removal in the Adsorption Clarifier is to relieve the solids loading on the Mixed Media filter. Use of coarse media, which has the advantages of high solids storage and low headloss, is desirable. This extends the run length of the filter, allowing the application of the process to relatively highly contaminated waters, while maintaining reasonable wastewater production. This should not be confused with simply adding a coarse media layer to the filtration process. There is a significant difference in that the clarifier is physically separate from the filter. This allows the unit to be cleaned independently of the filter. In addition, if particles the size of adsorption media were used in a filter, fluidization during backwash would require impractical flow velocities. By utilizing a separate clarifier with media that is buoyant, and with a specific gravity near that of water, a practical and inexpensive system can be employed which uses simple diffused air to fluidize the media. Fluidization kinetics are not reliant on fluid velocity. Flow reversal is not required allowing influent water rather than treated water to flush out solids.

This cleaning scheme utilizing raw water in the upflow mode allows the following benefits: 1) reduces chemical costs associated with using treated water for flushing, 2) simplifies piping and valving requirements, and 3) eliminates the need for separate pumps to flush the clarifier.

Another advantage of the separate, first stage clarifier, is that it does not have to be totally cleaned during a flush cycle. Clean bed headloss and treatment conditions are restored with a brief flush of 3 to 4 minutes at normal plant flowrates. In this amount of time, it is calculated that roughly 70 percent of the solids contained in the media are removed. The solids left in the bed serve as seed to promote rapid recovery of treatment efficiency.

Because of the economical flushing cycle, the wastewater from the clarifier is higher in solids concentration than that from the filter. Typical solids concentrations in the flush wastewater are in the range of 500 - 5,000 mg/L. They are readily settleable, reaching settled concentrations of 5,000 to 100,000 mg/L, depending on the characteristics of the raw water and the chemical treatment employed. The lower values are representative of colour, iron and manganese flocs and also low turbidity waters. These higher settled solid concentrations are typically seen in alum treatment of higher turbidity waters and when cationic polymers are used for coagulation.

ADSORPTION CLARIFIER AND INTEGRATED CLARIFIER/FILTER SYSTEM

Operation

A flow diagram indicating filtration and cleaning modes for the integrated system is provided in Figure 1.

During filtration, coagulating chemicals are added to the plant influent. The coagulated water then flows upward through the buoyant clarifier media, through a distribution trough into the filter section, and downward through a Mixed Media filter.

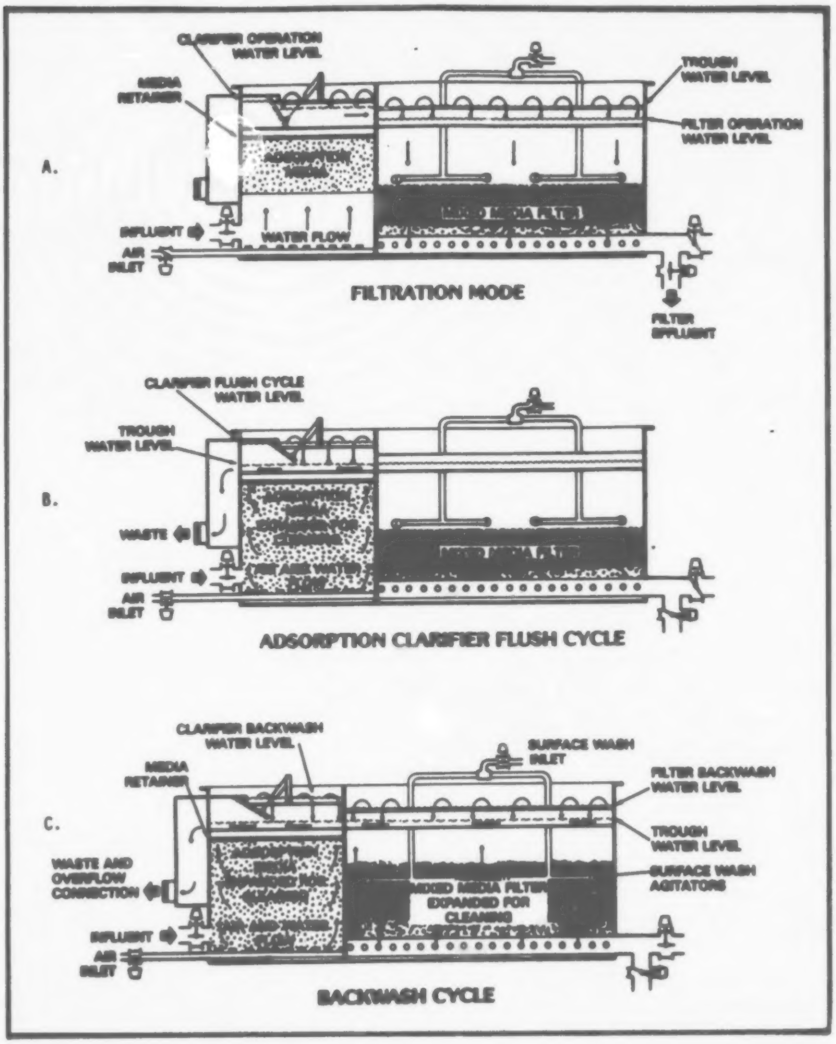
The adsorption media is cleaned by introducing diffused air into the bottom of the clarifier compartment to fluidize and scour the individual particles as shown in Figure 1B. The air causes an immediate expansion of the media. The air-water combination creates turbulence and has a lower density than water alone, thereby offsetting the buoyancy of the media. A vigorous scrubbing action takes place and solids are hydraulically flushed upwards to waste. This air-water scour system uses only influent water at the nominal influent flowrate and is not rate sensitive since the functions of fluidization and solids flushing are totally independent. The degree of solids flushing depends only on total wash volume. A total throughput of about 40 gal/sq. ft. is typically used. Large special pumps are not required, and the rate or duration need not be altered due to changing water temperature.

As a majority of the incoming solids are removed in the Adsorption Clarifier, flushing is normally required more often than backwashing of the mixed media filter. Appropriate controls allow intermediate flushing to occur automatically. During flushing, the Adsorption Clarifier retention system allows passage of waste solids but keeps the adsorption media in place. The retainer is scoured during each flush cycle, thus preventing clogging. It can also be easily removed from overhead to allow access to the compartment if necessary.

The mixed media filter is backwashed in the normal method as shown in Figure 1C at a nominal flow rate of 15-20 gpm/sq. ft. at 10-15°C. Surface scour is provided to assist in the backwashing process and elapsed time is about seven minutes.

Figure 1

Flow Diagram for Trident Water Systems


CLARIFIER OPERATION WATER LEVEL
MEDIA RETAINER
TROUGH WATER LEVEL
FILTER OPERATION WATER LEVEL
A.
MIXED MEDIA FILTER
INFLUENT
WATER FLOW
AIR INLET
FILTRATION MODE
FILTER EFFLUENT
CLARIFIER FLUSH CYCLE WATER LEVEL
TROUGH WATER LEVEL
B.
WASTE
INFLUENT
AIR INLET
ADSORPTION CLARIFIER FLUSH CYCLE
CLARIFIER BACKWASH WATER LEVEL
SURFACE WASH INLET
MEDIA RETAINER
FILTER BACKWASH WATER LEVEL
C.
TROUGH WATER LEVEL
WASTE AND OVERFLOW CONNECTION
SURFACE WASH AGITATORS
MIXED MEDIA FILTER EXPANDED FOR CLEANING
INFLUENT
AIR INLET
BACKWASH CYCLE

Performance

Table 1 shows that over forty sites have been piloted throughout the United States and Canada with the contact clarifier/filtration system.

Many full-scale installations have been installed and are running on waters where pilot studies were previously conducted. This allows direct comparison of results between the pilot equipment performance and the full-scale clarifier/filter systems. In all cases the chemical requirements, cleaning requirements and removals through the clarifier and filter were similar. This indicates that analysis of pilot data, which is usually more detailed than data from full-scale systems will reflect future full-scale performance.

Table A1 found in Appendix A shows an updated full-scale installation list which includes the plant location, capacity, and start-up date.

Table 1

List of Adsorption Clarifier/Mixed Media Filter System Pilot Studies

	Raw Water Character		
Location	Turbidity, NTU	Colour, Units	Comments
Corvallis, OR	45-120	5	
Rainier, OR	5	30	
Newport, OR	6-20	8	
Brockton, MA	3	55	
Newburyport, MA	1	45	
Greenfield, IA	5-8	15	Giardia removal
Palisade, CO	1	--	0.3-0.4mg/L manganese
Milton, WV	27-95	--	
Elkton, MD	7-17	--	
Petrolia, ON	2-24	1-9	
Charlevoix, MI	3	--	
Menominee, MI	2	15-20	Temp. = 1°C
FSC Paper; Alsip, IL	17-103	--	
Hingham, MA	1-3	110	
Bountiful, UT	5-20	--	Particle count
Vernal, UT	20	--	Particle count
Harrisburg, PA	3-170	--	
Ferriday, LA	30	--	
Eufala Lake, OK	35-50	--	
Dakota City, NE	--	--	5 mg/L iron
Sidney, MT	--	--	3 mg/L iron
100 Mile House, BC	0.7	--	Giardia removal
Rome, NY	0.5-4.9	35-85	Temp. = 1°C
Huntsville, TN	8	--	2-3 mg/L manganese
Redding, CA			
Blissfield, MI	14-380	25-240	
Tehachapi, CA	1-6	35-50	
St. Paul, Alberta	1.7-2.0	--	
Olathe, KS	12-18	--	
North Andover, MA	1	15	
Lake Texoma, TX	3-5	15-20	
Phillipsburg, PA	0.4-2.5	--	
Newport News, VA	3	30	
Wallingford, CT	.5-5	10	
Marin, CA	3-8	20	
Helena, MT	1-3	25	
Media, PA	1.0-70	--	Cold, clear water
Buffalo Pound, Sask, Canada	1-3	25	High dissolved gas content
Muskegon, MI	2-4	20	
Dover-Foxcroft, ME	.3-.5	20	Giardia removal
Meridith, NH	.3-.5	15	

Installations in Lewisburg, West Virginia, Greenfield, Iowa and Petrolia, Ontario have each been on-line for over two years. Summaries of the pilot and full-scale data for two of these installations are provided in Appendix B to show the data similarities.

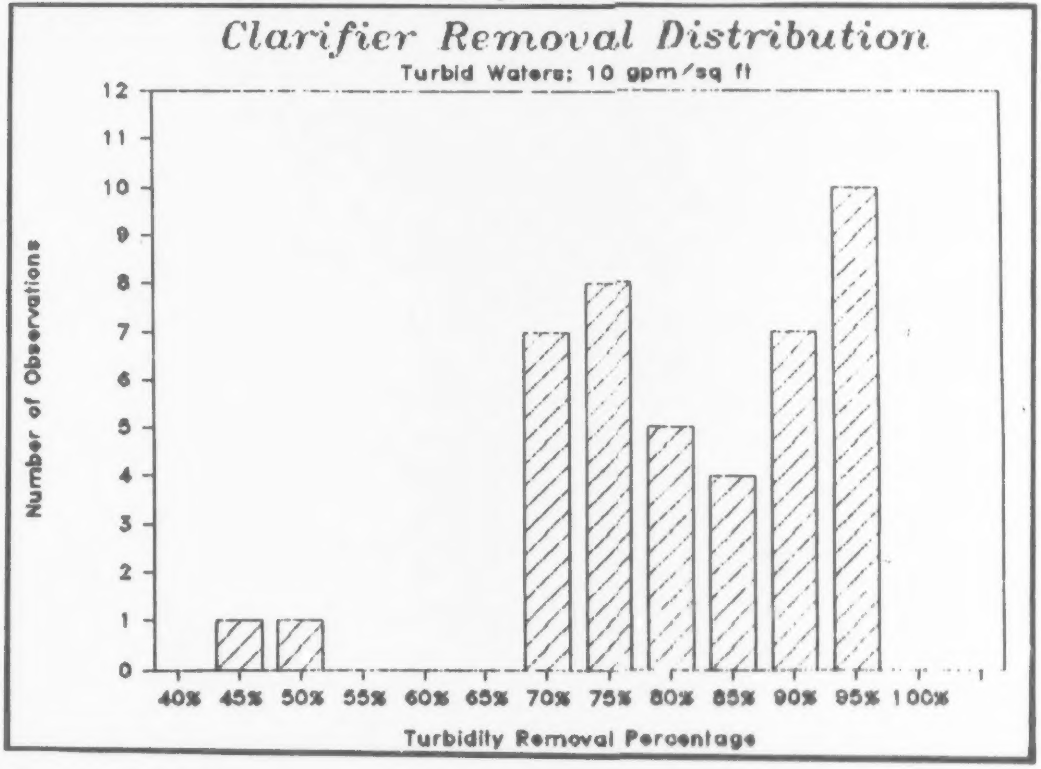
1. Turbidity Removal

Turbidity has been used as the primary measure of performance for the contact clarifier due to ease and convenience of measurement over other parameters. Figure 2 shows the clarifier turbidity removal distribution for 43 pilot study test runs.

Tables C1, C2, C3, and C4 from Appendix C, provide turbidity removal data from two full-scale installations. The clarifier performance also shown in Appendix C is presented in Figures C1, C2, C3 and C4. Clarifier turbidity removals at these sites averaged 80 percent. The average clarifier effluent turbidity was 0.97 NTU.

Figure 2

Clarifier Turbidity Removal Distribution



a. Applied Turbidity

Clarifier turbidity removal performance is based on comparison of the coagulated water applied to the clarifier with the clarifier effluent. Water contaminants, chemical type and chemical quantity can have a significant effect on turbidity applied to the clarifier.

The numerous pilot studies have shown that in turbidity removal applications alum and ferric salts provide an additional load to the clarifier whereas cationic polymers show little effect. In iron, manganese and colour removal applications, precipitated compounds cause a significant rise in clarifier influent turbidity. Flow rate, media depth, media characteristics, type of contaminant, type of chemical used, and natural water characteristics have been shown to affect the degree of turbidity removal in the contact clarifier.

b. Flowrate

Pilot testing at rates of 10 to 20 gpm/sq. ft. has shown minimal increase in effluent turbidity at higher clarification rates. This could be expected due to the higher velocities and shear forces within the clarifier media. Flowrate does not appear to be a critical design parameter in turbidity removal applications.

c. Media Depth

The turbidity removal efficiency has been shown to increase as media depth increases. However, the degree of improvement is less significant at depths over five feet. The optimum depth in terms of particulate removal performance, terminal headloss and equipment design was determined to be about 4 feet.

d. Media Characteristics

The smaller the effective clarifier media size the higher the particulate removal in addition to a higher head loss. Goals of maximum effluent quality with minimal down time for backwashing and wastewater generation must be balanced. Because of the Mixed Media filter back-up, clarifier media size does not appear to affect the effluent quality from the two stage clarifier/filtration process.

In relation to the cleaning operation, shape and specific gravity are critical to effective fluidization and solids removal.

e. Influent Characteristics

The nature of certain types of contaminants as well as their concentration can affect the turbidity removal efficiency. Figure 3 shows the relationship of clarifier influent turbidity to clarifier effluent turbidity from pilot data using alum addition. Table 2 shows the results of system effluent turbidity from 100 test runs at 20 pilot sites.

Figure 3

Comparison of Clarifier Influent Turbidity to Clarifier Effluent Turbidity

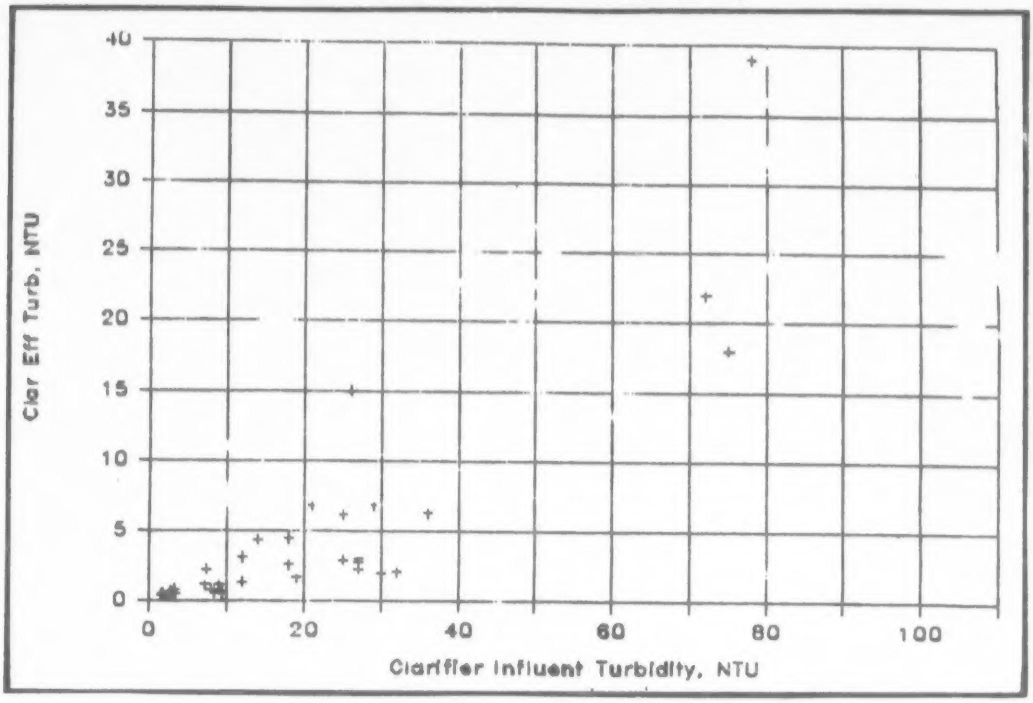


Table 2

Statistical Analysis of Effluent Turbidity Results from 20 TRIDENT® pilot studies

Parameter	Effluent Turbidity
Mean	0.24 NTU
Median	0.20 NTU
Mode	0.15 NTU
-1 Standard Deviation	0.13 NTU
+1 Standard Deviation	0.37 NTU

The data which supports this statistical summary indicates that system effluent qualities are not affected by raw water influent turbidity concentrations up to 100 NTU. Full-scale installations have operated successfully at peak turbidities of 470 NTU with little effect on effluent quality.

f. Chemical Addition

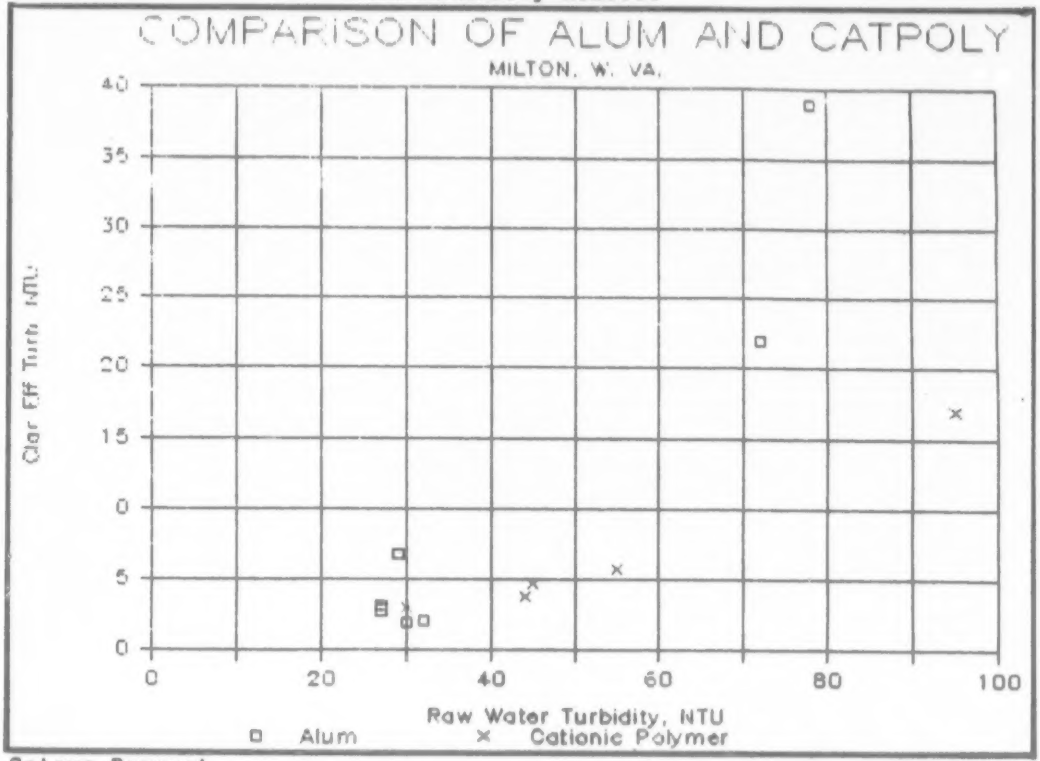
Chemical type and dosage can have an effect on clarifier performance in terms of run length and turbidity removal. The natural water characteristics also control the effectiveness of various types of chemical treatments.

The majority of municipal installations have used alum as the primary coagulant due to economics and handling. The basic process principles utilized with the upflow clarifier/filter system do not differ from conventional treatment. However, the upflow Adsorption Clarifier does provide the benefit of reduced coagulant requirements. This is because only a filterable pin floc has to be developed and not a large settleable floc.

Cationic polymers in place of alum have been used with success. Pilot study data from West Virginia shown in Figure 4 indicates comparable performance.

Figure 4

Comparison of Alum and Cationic Polymer on Turbidity Removal



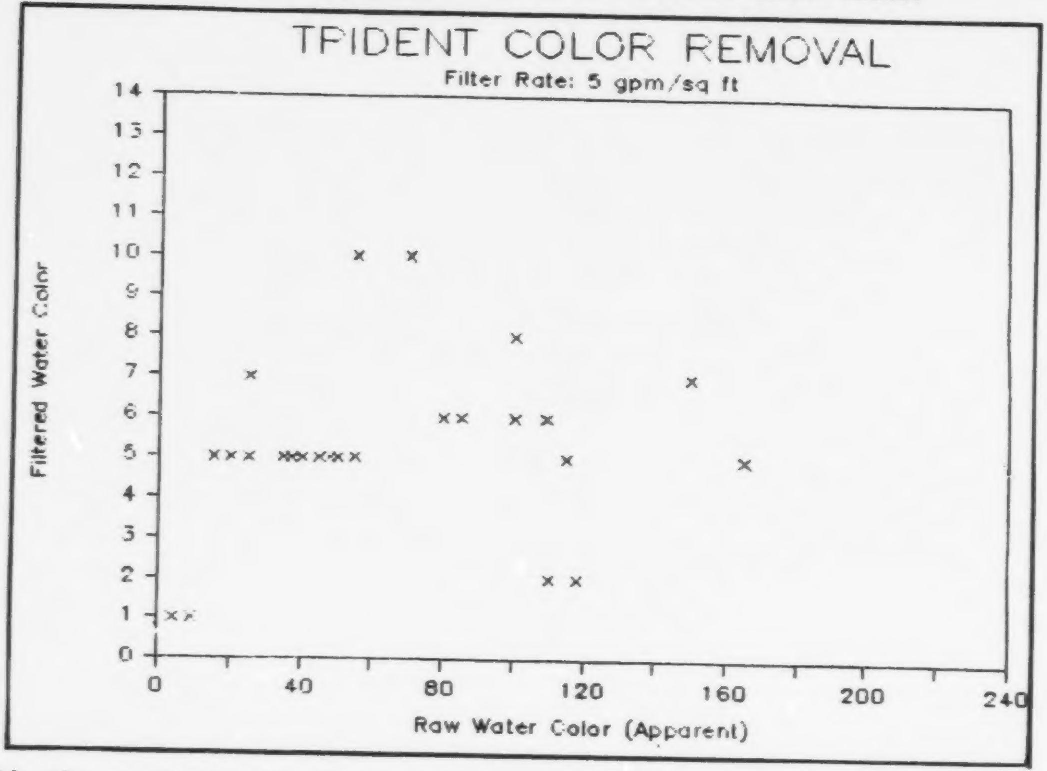
2. Colour Removal

During pilot studies, colour removal has not been measured through the Adsorption Clarifier. However, apparent colour was measured through the Adsorption Clarifier/Mixed Media filter system. Figure 5 shows the comparison of influent colour versus effluent colour for nine pilot sites where colour removal was the main focus of the study. This figure indicates that with proper chemical addition, the colour removal standard of 15 cu can be met with influent colour concentrations up to 240 apparent cu.

The Adsorption Clarifier provides a significant benefit in colour removal applications. A colour floc is typically very light and hard to settle. High colour often occurs during late autumn months with correspondingly colder water temperatures which have the effect of hindering settling.

Figure 5

Comparison of Raw Water Colour To Filtered Water Colour



With the contact clarifier settling is not required. Colour causing constituents are coagulated and removed within the clarifier media.

During pilot studies at Rome, N.Y. influent colours of 110 cu were reduced to 5 cu with temperatures less than 1^{o}C. Colour removal was accomplished through conventional alum coagulation.

3. Iron and Manganese Removal

Three pilot studies have been conducted specifically for iron and manganese removal. These studies have shown that the Adsorption Clarifier/Mixed Media system is very effective in these applications. Two of these pilot studies led to the installation of full-scale plants.

In a system such as the Adsorption Clarifier/Mixed Media filter unit iron removal by precipitation and filtration is achieved. Iron concentrations of up to 8 mg/L have been reduced to below 0.3 mg/L by oxidation with either potassium permanganate or chlorine.

The upflow contact clarifier provides two additional benefits in iron and manganese applications:

1. Like colour floc, iron floc is typically light, fluffy and hard to settle. The use of an Adsorption Clarifier eliminates the need for extensive settling time and large basin capacity.

2. Literature suggests that "hydrous oxides of Fe (III) and manganese dioxide both have high sorption capacities for Fe (II) and Mn (II)" (6). Thus floc particles trapped during the initial portion of a run may further enhance iron and manganese removal during the remainder of the run.

4. Particle Counts and *Giardia* Removal

Particle counts have been measured during several different pilot studies. Particles removals in the 5 to 12 micron range (*Giardia* cyst size) were of particular interest at six studies in New York, Colorado, Utah, Maine and British columbia. Particle count removals were reported to be 96 to 98 percent in the 5 to 15 um size range and 98 to 99 percent in the 5 to 10 um size range at two of the pilot sites. No influent counts were measured at two of the studies, therefore, no removals are indicated. However, effluent particle levels in all size ranges were reported to be consistently low. At another pilot study, effluent samples were analyzed for *Giardia* cysts. No cysts were detected.

On a full-scale basis to date, the contact clarifier/Mixed Media filtration system has been installed at several locations to combat the threat of *Giardia* outbreaks. With proper chemical addition and operation, the two stage/clarification filtration system has proven to be very effective in *Giardia* removal.

5. Washwater Volume

Net production and wastewater volumes are affected by the level of contaminants and their nature as well as chemical dose. Both pilot study and full-scale data indicate that wastewater volume generally ranges from 2 to 5 percent of net production capacity. Typical values for backwash volume in conventional treatment applications range from 3 to 8 percent (7).

COMPARISON TO ALTERNATE CLARIFICATION DESIGNS

The contact clarifier offers several advantages over other processes currently used for flocculation/sedimentation. One advantage is performance; the contact clarifier performs well in the removal of fine, light and delicate flocs that are formed in coloured and low turbidity applications because this solids/liquid separation process is not dependent on gravity. Another advantage is cost; to achieve the same suspended particulate removal as other processes a smaller less costly contact clarifier can be employed. Table 3 compares surface area requirements for various solids removal processes.

The table shows that even without considering the flocculation step for conventional sedimentation, tube settlers, and plate separators, the Adsorption Clarifier plan area is the smallest effective unit.

A closer comparison of the 2 mm sand filter and the contact clarifier is shown in Figure 6. This figure was taken from a study conducted in Redding, California by engineering consultants CH2M-Hill. They show that the rate of headloss increase through the contact clarifier was lower for the entire run length than the 2 mm sand filter. Run time to 10 feet of headloss was over 30 percent longer with the contact clarifier. In addition to lower headloss, this study showed that the contact clarifier had a consistently lower effluent turbidity and higher solids capture rate.

Table 3

Plan Area Comparison of Solids Removal Equipment

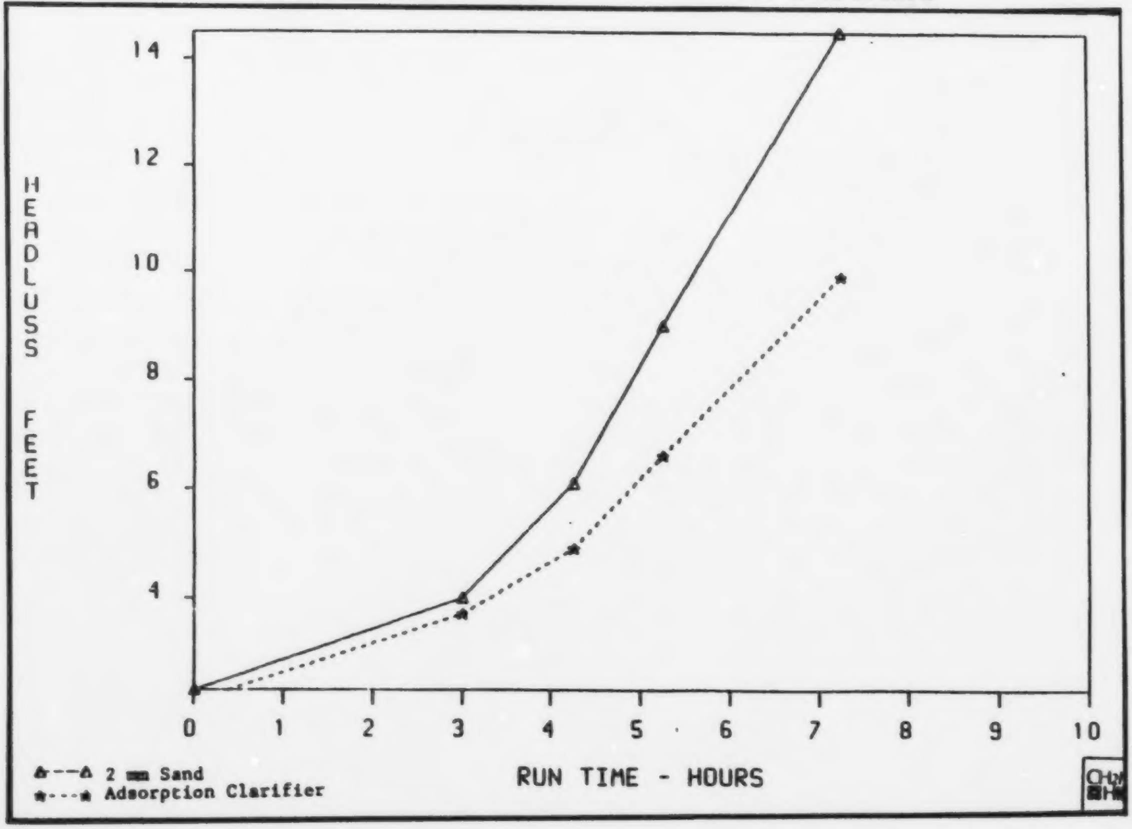
METHOD	RATE	PLAN AREA
1. Adsorption Clarifier^{tm}	10 gpm/ft^2	70 ft^2
2. 2 mm Sand Filter	7 gpm/ft^2	100 ft^2
3. Plate separator*	3.7 gpm/ft^2	188 ft^2
4. 60^0 Tube Clarifier*	2 gpm/ft^2	384 ft^2
5. Conventional Flocculation Sedimentation*	800 gpd/ft^2	1250 ft^2

Assume 1 MGD plant flowrate (695 gpm)

*Plan area for flocculation not included.

Figure 6

Relative Headloss Development for Contact Clarifiers



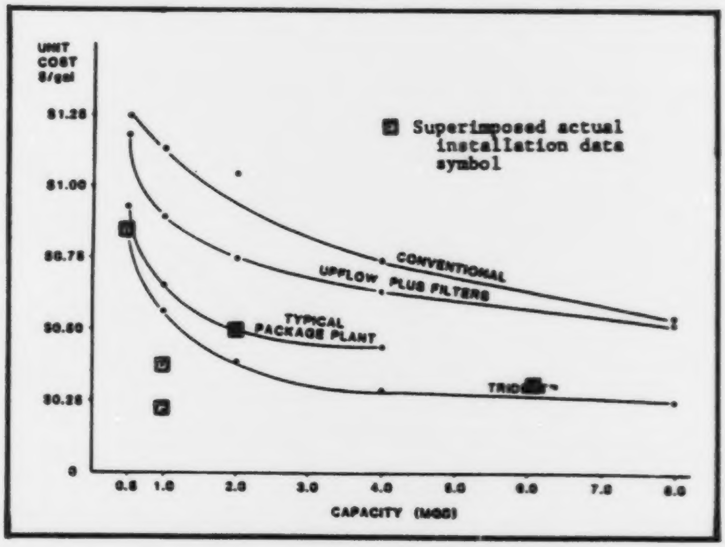
COMPLETE TREATMENT COST ANALYSIS

In 1983 an independent consultant [12] conducted a unit cost analysis for four treatment alternatives. These included conventional design [coagulation, flocculation sedimentation and filtration], upflow clarifiers with gravity filters, typical package plants, and Adsorption Clarifier/filtration process [Trident®]. The results of this study are shown in Figure 7. The Adsorption Clarifier/filtration process was the most cost effective system. In the 0.5 to 8.0 mgd flow range this process offers savings from 36% up to 89%.

Included in the cost estimates are treatment equipment, chemical feed systems, buildings, building foundations, piping, valves, instrumentation, engineering and administration. Because of uncontrollable variations, the costs of land, high service pumping, water intakes, and any unusual requirements were excluded from the cost analysis.

[illegible]gure 7

Unit Costs for Treatment Alternatives



To support the validity of this study, the actual costs from four recent full-scale installations are superimposed on the curve in Figure 7 for the Adsorption Clarifier/filtration process. As predicted, the actual costs for this process are well below other treatment alternatives.

In June of 1985, this study was expanded to include flow ranges from 8 to 20 MGD. Processes studies included conventional treatment facilities, conventional plant utilizing tube settlers in clarifiers, conventional plant utilizing solids contact clarifier systems, conventional plant utilizing lamella plates in the clarifiers, and the Adsorption Clarifier/filtration process designed in concrete [also known as Tricon®].

Scope of supply paralleled the earlier study. In the 8 to 20 MGD flow range the Tricon® consistently was more cost effective than all other options. Tables 4 and 5 show the results from this study.

SUMMARY

The innovation of the contact clarifier has allowed a new integrated water treatment process to be developed. Substantial space and cost savings are made possible by combining the flocculation and clarification unit operations into a simple high-rate upflow process. Process capabilities have been and continue to be demonstrated in numerous demonstration and full-scale installations. Applications include turbidity, colour, iron and manganese removal. Suitability of this process in the reduction of Giardia size particles has also been shown. Cost economics over a flow range of 0.5-20 mgd show substantial savings over conventional and alternative high rate systems. The integrated contact clarifier/filter system provides not only excellent performance but also at the lowest cost.

Table 4

Estimated Capital Costs

		Plant Type			
Flow MGD	Conventional	Conventional w/ Tube Settlers	Conventional w/ Solids Contact	Lamella Plates	TRICON
8	5,089,000	4,245,000	4,390,000	4,407,000	3,756,000
12	6,606,000	5,818,000	5,768,000	5,781,000	4,883,000
15	7,699,000	6,572,000	6,986,000	6,746,000	5,559,000
20	9,235,000	8,307,000	8,382,000	8,006,000	6,938,000

Table 5

Cost Comparisons per Gallon Treated

		Plant Type			
Flow MGD	Conventional	Conventional w/ Tube Settlers	Conventional w/ Solids Contact	Lamella Plates	TRICON
8	.64	.53	.55	.55	.47
12	.55	.48	.48	.48	.41
15	.51	.44	.47	.45	.37
20	.46	.42	.42	.40	.35

ACKNOWLEDGEMENT

Microfloc Products gratefully acknowledges the valuable assistance of the plant operating personnel at each of the full scale TRIDENT® and Adsorption Clarifier$^{\text{tm}}$ installations who provided the performance data utilized in this report.

Their dedication and expertise in providing a safe potable water is a credit to both themselves and the profession.

Additionally we thank those municipalities and water districts who have assisted our pilot program by providing facilities and services to run over 40 individual studies across the U.S. and Canada.

REFERENCES

1. Culp, G.L., and Culp, R.L., New Concepts in Water Purification, Van Nostrand Reinhold, New York, 1974.
2. Adin, A., and Rehbun, M., "High Rate Contact Flocculation - Filtration with Cationic Polyelectrolytes," Journal American Water Works Association, p. 109 (1974).
3. Saleh, F., and Cleasby, J., "Kinetics of Bed Contact Flocculation," Proceedings of 1983 National Conference on Environmental Engineering, p. 125.
4. Ives, K.J., "Coagulation and Flocculation; Particle Flocculators," Chapter from Solid/liquid separation, edited by Svarovsky, p. 107 (1975).
5. Treweek, G.P., "Optimization of Flocculation Time Prior to Direct Filtration," Journal AWWA, p. 96 (1979).
6. O'Connor, J.T., "Iron and Manganese," Chapter 11 of Water Quality and Treatment, AWWA (1971).
7. Sanks, R.L., Water Treatment Plant Design, Ann Arbor Science, 1978.
8. Benjes, H.H. Jr., Edlund, C.E., and Gilbert, P.T., "Adsorption Clarifier Applied to Low Turbidity Surface Supplies," from AWWA Proceedings, June 1985, Washington D.C.
9. Hemphill, B.W., and Belyea, A.R., "Field Experience With Two-Stage Contact Flocculation/Filtration System," Presented at Aqua Tech Conference, Los Angeles, CA, 1984.
10. Mueller, H.M., "An Innovative Water Treatment Process," Presented at Kentucky - Tennessee AWWA - WPCF Conference, Louisville, KY, November 1983.
11. "High-Tech Water in the Green Fields of Iowa, "Water/Engineering and Management, January, 1985.
12. "Summary of Cost Estimates for Various Water Treatment Plants," Owen Engineering and Management, May 1983.
13. "Summary of Cost Estimates for Various Water Treatment Plants," Owen Engineering and Management, June 1985.

APPENDIX A

Full Scale Installation List

Table A1. Adsorption Clarifier/Mixed Media Filter Installation List

PARTIAL LIST OF INSTALLATIONS

TRIDENT™ WATER SYSTEMS

Name/Location	Job#	Model	Capacity	Startup
Lewisburg, West Virginia	P-5934	TR-420 with AQUARITROL®	2.0 MGD	Dec. 1983
Colorado City, Colorado	P-5963	TR-210	1.0 MGD	Nov. 1983
Red Lodge, Montana	P-5968	TR-420 with AQUARITROL®	1.4 MGD	Jan. 1984
FSC Paper Corporation Alsip, Illinois	P-5972	2 TR-840 with AQUARITROL®	8.0 MGD	Mar. 1984
Greenfield, Iowa	P-5979	1 1/2 TR-210 with AQUARITROL®	1.0 MGD	Apr. 1984
Temagami, Ontario, Canada	B-1229	1/2 TR-105 with AQUARITROL®	0.25 MGD	May 1984
Petrolia, Ontario, Canada	B-1231	TR-420 with AQUARITROL®	2.0 MGD	Apr. 1984
Hot Sulphur Springs, Colorado	P-5998	TR-105 with AQUARITROL®	0.5 MGD	May 1984
Melville, Saskachewan, Canada	B-1233	TR-420 with AQUARITROL®	2.0 MGD	---
East Carbon City, Utah	P-6005	TR-210 with AQUARITROL®	1.0 MGD	July 1984
Dauphin Consolidated Water Supply, Co. Harrisburgh, Pennsylvania	P-6011	3 AC-140 with AQUARITROL®	9.0 MGD	Apr. 1984
Iowa Beef Processors Dakota City, Nebraska	P-6013	TR-420	2.0 MGD	May 1984
Ashley Valley WTP Vernal, Utah	P-6022	1 1/2 TR-840 with AQUARITROL®	6.0 MGD	Aug. 1985
Manitou Springs, Colorado	P-6024	1 1/2 TR-420 with AQUARITROL®	3.0 MGD	June 1984
Ft. Washakie, Wyoming	P-6028	TR-210A with AQUARITROL®	1.0 MGD	---
Parkville Water District Leadville, Colorado	P-6029	TR-420 with AQUARITROL®	2.0 MGD	Jan. 1985
Nanton, Alberta, Canada	N-9800	1/2 TR-210A	0.5 MGD	April 1985
Sneedville, Tennessee	P-6032	1/2 TR-210 A	0.5 MGD	May 1985
Huntsville, Tennessee	P-6039	TR-420 with AQUARITROL®	2.0 MGD	May 1985
Pagosa Springs, Colorado	P-6044	1/2 TR-210A with AQUARITROL®	0.5 MGD	---
North Greene, Tennessee	P-6053	TR-105A with AQUARITROL®	0.5 MGD	Aug. 1985
Champion International Ontonagon, Michigan	P-6055	TR-210A with AQUARITROL®	0.6 MGD	April 1986
Greenville, West Virginia	P-6056	TR-210 with AQUARITROL®	1.0 MGD	Aug. 1985
Rome, New York	P-6060	4 TR-840A	15.0 MGD	--
Delores, Colorado	P-6061	AC-70	1.0 MGD	July 1985

Table A1. Continued

Donala Colorado Springs, Colorado	P-6063	TR-210A with AQUARITROL®	1.0 MGD	Sept. 1985
Crested Butte, Colorado	P-6067	TR-420A with AQUARITROL®	2.0 MGD	Feb. 1986
Baggs, Wyoming	P-6068	AC-35	0.5 MGD	Aug. 1985
Frisco, Colorado	P-6069	TR-420A with AQUARITROL®	2.0 MGD	March 1985
Philomath, Oregon	P-6070	TR-210 with AQUARITROL®	1.0 MGD	Feb. 1986
Montezuma Delores, Colorado	P-6076	TR-420 with AQUARITROL®	2.0 MGD	Dec. 1985
Spring Hill, Kansas	P-6078	TR-210A with AQUARITROL®	1.0 MGD	March 1986
Pagosa Springs, Colorado	P-6083	TR-420 with AQUARITROL®	2.0 MGD	March 1986
Edgemont Ranch Durango, Colorado	P-6086	1/2 TR-210 with AQUARITROL®	0.5 MGD	Nov. 1985
Arkins Water District Loveland, Colorado	P-6087	AC-35	0.5 MGD	--
Longtown, Oklahoma	P-6088	TR-420 with AQUARITROL®	2.0 MGD	March 1986
Bountiful, Utah	P-6090	TR-420A with AQUARITROL®	2.0 MGD	---
Palm Springs WWTP California	P-6093	1 1/2 TR-840A with AQUARITROL®	5.0 MGD	---
Mountain City, Tennessee	P-6095	TR-210A with AQUARITROL®	1.0 MGD	---
Rock Springs, Wyoming	P-6103	TR-420 with AQUARITROL®	2.0 MGD	---
Thompson Creek Kent Island, Maryland	P-6104	TR-105A with AQUARITROL®	0.5 MGD	---
Buffalo Narrows Saskatchewan, Canada	P-6106	1/2 TR-105A with AQUARITROL®	0.25 MGD	---
Canora, Saskatchewan, Canada	P-6107	1 1/2 TR-210A with AQUARITROL®	1.5 MGD	---
Scurry Rainbow Oil Ltd. Ft. St. John, B.C., Canada	P-6108	1/2 TR-105A	0.25 MGD	---
Wynyard, Saskatchewan, Canada	P-6115	TR-210A with AQUARITROL®	1.0 MGD	---
Ute Pass Colorado Springs, Colorado	P-6116	TR-210 with AQUARITROL®	1.0 MGD	---
Hay River, Alberta, Canada	P-6117	TR-210A with AQUARITROL®	1.0 MGD	---

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APPENDIX B

Pilot and Full Scale Operating Data

Table B-1

Summary of Operating Conditions and Results

Greenfield IA, PILOT STUDY

		Chemical Feed, [illegible]					Turbidity, NTU ③				Alkalinity, mg/l		pH	
Run	Dates	Alum	Filter① Aid Poly.	Coag.② Poly.	Cl_2	Activ. Carbon	Raw	Clar. Inf.	Clar. Eff.	Filter Eff.	Raw	Final	Raw	Final
1	1/20	41	0.1	0	0	0	7.8	12.9	5.2	0.35	170	135	7.6	6.5
2	1/21-1/22	30	0	0.5	0	0	7.2	10.5	3.9	0.46	165	135	7.7	6.6
3	1/24	30	0	0.5	0	0	7.2	11.0	4.4	0.40	170	150	7.3	6.8
4	1/25-1/26	31	0.02-0.1	0.5	0	0	7.1	10.2	3.6	0.40	170	150	7.2	6.8
5	1/26-1/27	30	0.1-0.2	0.5	0	0	6.6	10.2	2.8	0.39	170	150	7.4	6.8
6	1/28-1/29	20	0.1	0-0.5④	0	5	6.9	10.9	3.4	0.50	170	150	7.4	6.8
7	1/31	31	0.1	0	1.0	0	9.4	13.8	4.9	0.48	170	150	7.3	6.8
8	2/1	25	0.1	0	0	0	8.1	11.4	3.6	0.40	170	150	7.4	6.8

--All runs were at 10 gpm/sq.ft. on clarifier and 5 gpm/sq.ft. on filter.
--Water temperature: 40°-41°F.
--Raw water color approximately 15 units.

Notes:

① Filter aid polymer was American Cyanamid No. 985N, nonionic.
② Coagulant polymer was Nalco 8103, cationic.
③ Turbidity shown is mean value of hourly grab samples, measured in Hach 2100A.
④ Turned off halfway through run.

Table B-2

Full Scale Operating Data
Greenfield Iowa, 1.5 MGD TRIDENT

Week Beginning:	1 July	8 July	15 July	22 July	29 July	5 Aug	12 Aug	19 Aug	26 Aug	2 Sept	9 Sept	16 Sept
Gallons Treated x 1,000	2,444	2,303	2,356	2,200	2.161	2,432	2,257	2,003	2,233	1,853	1,947	2,031
Raw NTU Ave.	7.84	7.36	7.13	4 17	2.85	3.24	3.47	3.54	5.4	4.33	3.95	4.81
Range	6.5-12	6.0-8.3	4.4-13.0	3.4-6.3	2.5-5.2	2.7-4.2	2.5-4.1	2.7-4.0	4.0-6.0	3.6-5.5	3.8-4.0	4.4-5.3
Clar. Eff.	—	--	—	—	—	1.27	1.15	1.57	1.08	1.04	1.55	2.3
Filter Eff.	0.37	0.52	0.52	0.54	0.60	0.54	0.46	0.47	0.37	0.40	0.38	0.38
Alum lbs.	400	300	350	250	200	300	450	150	200	200	100	100
ppm	19.6	15.6	17.8	13.6	11.1	14.8	23.9	8.9	10.7	12.9	6.2	5.99
Polymer lbs.	7	6	6	6 1/2	5	7 1/2	5	8	3	5	6	6
ppm	.34	.31	.31	.35	.28	0.37	0.26	0.48	0.16	0.32	0.37	0.36
Soda lbs.	50	50	0	50	0	0	100	0	0	50	0	50
ppm	2.45	2.6	—	2.7	—	—	5.3	—	—	3.24	—	2.99
Cl_2 lbs.	136	142	118	120	156	114	44	38	101	125	132	123
ppm	6.67	7.4	6.0	6.5	8.7	5.62	2.34	2.27	5.42	8.09	8.13	7.37
Carbon lbs.	—	--	—	—	—	—	50	—	—	—	—	—
ppm	—	—	--	--	—	—	2.65	—	--	--	--	—

Table B-3
Petrolia Ontario, 2 MGD TRIDENT
Pilot Study Data

SUMMARY OF OPERATING CONDITIONS AND RESULTS

Run	Date	Rate gpm/ft2		Chemical Feed, mg/L			Turbidity NTU								
							Raw			Clarifier			Final		
		Clarifier	Filter	Alum	Poly	Cl2	High	Avg.	Low	High	Avg.	Low	High	Avg.	Low
1	12/2-12/3	10	5	13.2	0	0	9.0	5.9	5.2	2.9	1.2	0.42	0.53	0.17	0.10
2	12/3-12/4	10	5	10.6	0.047	0	4.1	1.9	1.2	3.4	0.55	0.20	0.31	0.13	0.07
3	12/5-12/6	15	7.5	10.6	0.085	3.8	8.9	5.2	2.7	3.1	1.0	0.45	0.79	0.16	0.08
4	12/6	15	7.5	10.6	0.085	3.8	20.0	9.7	4.1	2.5	1.1	0.23	0.20	0.13	0.05
5	12/7	20	10	12.9	0.082	3.8	9.8	8.1	6.9	2.2	1.4	0.55	0.20	0.15	0.10
6	12/8-12/9	10	5	12.0/16.0	0.095	3.5/0	75.0	24.4	6.0	12.0	2.02	0.49	0.28	0.14	0.11

Table B-3 continued
Petrolia Ontario, 2 MGD TRIDENT
Pilot Study Data

HEADLOSS DEVELOPMENT RATES AND EFFICIENCY PROJECTIONS

Run	Rates, gpm/sq.ft.		Clarifier		Mixed Media Filter		Net Production Percent	Backwash Waste % Flow
	Clarifier	Filter	Headloss Rate, ft/hr	Run Length Hours	Headloss Rate, ft/hr	Run Length Hours		
1	10	5	0.23	14.4	0.13	50.0	98.3	1.2
2	10	5	0.30	11.0	0.17	38.2	97.8	1.5
3	15	7	0.53	4.9	0.23	26.1	96.3	2.4
4	15	7.5	0.70	3.7	0.20	30.0	95.6	2.7
5	20	10	0.78	[illegible].6	0.64	6.9	91.8	5.3
6	10	5	0.38	8.7	0.14	46.4	97.7	1.6

Basis for Projections

1. Run lengths are computed for a terminal headloss of 4 ft. on the clarifier and 8 ft. for the filter.
2. Total volume is 40 gal/sq. [illegible]r each clarifier flush; and 100 gal/sq.ft. for each Mixed Media flush.
3. Total down time for each flush is 7 minutes; for each backwash is 10 minutes.

Turbidity Data Sept-Dec

Table B-4. Full Scale Data, 2 MGD TRIDENT

Petrolia Ontario

Petrolia Water Plant Turbidities

Plant Data [illegible]

September

Raw Water

Day	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31
6 AM						35	23	7	2	2	1	1	2	1	22	17	7	10	6	5	38	15	4	2	2	80	70	42	22	25	
8						34	22	7	1	1	1	4	2	5	23	28	8	7	7	4	42	13	3	6	1	77	58	41	28	21	
10						31	20	8	1	1	1	4	1	14	27	32	10	5	7	2	43	12	2	1	1	85	58	38	32	21	
12						29	17	4	1	2	1	5	1	23	29	31	10	5	7	2	48	11	5	1	1	101	52	37	38	22	
2 PM							16	4	1	1	1	4	1	23	32	32	20	5	7	1	41	4	2	2	1	92	54	34	47	19	
4						21	14	2	1	1	1	3	1	25	29	33	13	6	11	4	48	5	2	1	1	67	42	32	58	17	
6						21	14	2	1	1	1	2	1	21	35	32	15	6	8	7	34	3	2	1	1	88	33	38	38	18	
8					35	23	9	2	1	1	1	2	1	20	25	17	18	7	8	8	23	3	2	1	1	72	33	38	25	14	
10					42	21	8	2	2	1	1	2	1	46	27	7		7	8	10	18	3	3	1	1	60	42	27	23	13	
12					51	23																									
2 AM					43																										
4					36																										
6					27																										
No.					6	9	9	9	9	9	9	9	9	9	9	9	9	9	9	9	9	9	9	9	9	9	9	9	9	9	
Ave.					40	26.44	15.89	4.22	1.22	1.22	1.00	3.11	1.22	20.07	28.70	25.22	13.12	6.44	7.44	4.78	35.22	7.67	2.78	1.78	1.11	78.44	48.50	34.67	32.78	18.67	
Max.					51	35	23	8	2	2	1	5	2	46	35	33	20	10	11	10	48	15	5	8	2	101	70	42	50	25	
Min.					27	21	8	2	1	1	1	2	1	21	25	7	13	5	6	1	18	3	2	1	1	60	33	27	23	13	
Diff. Max-Min					24	14	15	6	1	1	0	3	1	24	10	26	7	5	5	9	30	12	3	5	1	41	37	15	27	12	

Totals

228No. Read

17.78Ave Read

101.00Max Read

1.00Min Read

41.00Max.D Read

Treated Water

September

Day	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31
6 AM						.53	.47	.64	.50	.70	.64	.62	.58	.59	.55	.55	.64	.61	.65	.55	.63	.59	.60	.6	.6	.60	.7	.56	.57	.53	
8						.49	.57	.61	.50	.50	.50	.50	.54	.6	.60	.58	.50	.54	.56	.62	.57	.55	.6	.59	.6	.60	.5	.50	.58	.58	
10						.54	.50	.58	.58	.6	.6	.59	.55	.55	.64	.58	.50	.62	.6	.6	.67	.54	.58	.6	.6	.60	.50	.50	.6	.50	
12						.55	.50	.61	.50	.6	.6	.61	.50	.65	.54	.58	.56	.64	.61	.50	.6	.55	.56	.7	.6	.60	.6	1.01	.6	.70	
2 PM						.50	.6	.6	.6	.57	.61	.6	.61	.60	.6	.61	.55	.6	.50	.54	.59	.59	.61	.6	.6	.62	.6	.54	.61	.6	
4						.55	.50	.50	.6	.6	.65	.6	.65	.67	.50	.60	.50	.6	.6	.65	.50	.65	.65	.55	.60	.65	.6	1.15	.60	.59	
6						.60	.61	.60	.6	.6	.64	.65	.66	.61	.54	.57	.55	.64	.56	.61	.59	.63	.64	.6	.75	.60	.70	.64	.6	.64	
8					.75	.65	.64	.6	.66	.6	.59	.65	.6	.5	.58	.6	.50	.58	.7	.62	.59	.61	.60	.58	.74	.65	.63	.62	.6	.55	
10					.7	.61	.64	.6	.6	.6	.6	.6	.6	.62	.55	.58	.6	.59	.58	.64	.6	.61	.7	.6	.77	.60	.61	.61	.61	.6	
					.71																										
					.78																										
					.78																										
					.78																										
No.					6	9	9	9	9	9	9	9	9	9	9	9	9	9	9	9	9	9	9	9	9	9	9	9	9	9	
Ave.					.74	.57	.58	.61	.60	.64	.61	.61	.60	.60	.58	.58	.58	.65	.61	.60	.60	.60	.65	.60	.60	.60	.71	.70	.63	.60	
Max.					.78	.65	.64	.65	.68	.7	.65	.65	.65	.67	.60	.60	.64	.64	.7	.66	.67	.63	.6	.7	.77	.62	.80	1.15	.60	.70	
Min.					.7	.48	.47	.58	.58	.57	.58	.58	.54	.5	.54	.55	.50	.54	.50	.54	.57	.54	.56	.59	.6	.60	.6	.54	.57	.53	
Diff. Max-Min					.08	.17	.17	.07	.07	.13	.08	.08	.12	.17	.12	.14	.08	.1	.14	.12	.1	.15	.24	.14	.17	.16	.30	.61	.32	.25	

Totals

231No. Read

.60Ave Read

1.15Max Read

.47Min Read

.68Max.D Read

Turbidity Data Sept-Page 2

Table B-4. Full Scale Data, 2 MGD TRIDENT

Petrolia Untario

October

Raw Water

Day	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	Total Month	
6 AM	12	15	9	51	35	18	7	7	8	5	4	5	4	2	1	1	2	2	9	3	2	3	9	5	4	6	2	3	30	20	61		
8	13	12	5	70	32	21	7	8	7	5	4	2	5	2	1	1	1	3	7	2	8	7	5	8	4	5	2	5	23	19	79		
10	14	8	8	63	40	20	7	8	9	4	3	2	3	2	1	1	1	5	8	2	8	14	7	5	4	4	2	3	34	14	100		
12	18	8		58	34	20	6	8		4	3	2	3	2	1	1	1	12	5	2	8	16	8	5	4	4	2	3	33	14	117		
2 PM	15	9	5	45	44	19	6	6	8	5	3	3	3	1	1	1	1	22	4	2	4	12	7	3	3	3	2	3	44	18	101		
4	14	9		41	35	24	5	6	6	4	3	3	2	1	2	2	1	21	5	2	3	13	6	4	3	2	1	2	36	13	100		
6	16	8		40	30	16	5	6	6	4	5	3	2	1	2	1	1	17	5	1	3	11	5	4	3	2	1	2	34	16	119		
8	18	8	21	35	30	20	5	6	6	4	5	3	2	1	1	2	1	14	4	1	3	9	4	4	3	2	1	2	30	13	58		
10	20	7	60	31	35	18	6	6	6	3	4	3	2	1	2	1	1	14	3	2	4	9	4	4	3	2	2	5	27	9	43		
12	21	7	34	30	22	15			5	4	8	2	2	1	2	1	1	13	3	2			4	3			2	7	19	11	44		
2 AM			35																														
4			50																														
6																																	
No.	10	10	9	10	10	10	9	9	9	10	10	10	10	10	10	10	10	10	10	10	9	9	10	10	9	9	10	10	10	10	10	302	No.
Ave.	16	9.00	25.78	46.70	33.70	19.10	6.00	6.89	6.89	4.20	4.20	2.80	2.80	1.40	1.40	1.20	1.10	12.30	5.30	1.90	4.11	10.44	5.90	4.50	3.44	3.33	1.70	3.50	31.40	13.80	81.00	11.07	Ave.
Max.	21	15.00	60.00	70.00	44.00	24.00	7.00	8.00	9.00	5.00	8.00	5.00	5.00	2.00	2.00	2.00	2.00	22.00	9.00	3.00	8.00	16.00	9.00	8.00	4.00	6.00	2.00	7.00	44.00	20.00	117.00	117	Max.
Max/Ave	1.31	1.67	2.33	1.50	1.31	1.26	1.17	1.16	1.31	1.19	1.90	1.79	1.79	1.43	1.43	1.67	1.82	1.79	1.70	1.58	1.46	1.53	1.53	1.64	1.16	1.80	1.18	2.00	1.40	1.45	1.44	9.05	

October

Treated Water

Day	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	Total Month	
6 AM	.40	.01	.01	.60	.6	.60	.63	.6	.6	.6	.58	.5	.53	.54	.55	.54	.58	.55	.62	.6	.59	.59	.73	.55	.64	.54		.56	.58	.58	.50		
8	.55	.50	.6	.63	.65	.64	.74		.6	.6	.58	.52	.54	.54	.58	.53	.58	.53	.61	.58	.55	.54	.56	.55	.54	.55	.55	.50	.58	.75	.56		
10	.55	.63	.6	.65	.64	.65	.59	.61		.50	.58	.52	.54	.6	.56	.50	.58	.53	.57	.56	.56	.55	.55	.58	.55	.55	.58	.53	.54	.54	.58		
12	.53	.62		.64	.65	.64	.73	.6		.58	.51	.52	.7	.54	.54	.50	.53	.54	.64	.54	.56	.55	.54	.55	.54	.58	.56	.55	.54	.58	.58		
2 PM	.50	.72	.6	.61	.60	.64	.6	.6	.40	.55	.52	.53	.64	.55	.58	.54	.54	.54	.6	.54	.54	.55	.58	.55	.54	.54	.56	.56	.58	.55	.61		
4	.55	.6		.50	.65	.64	.62	.6	.55	.50	.5	.55	.54	.55	.52	.55	.53	.54	.55	.53	.53	.58	.54	.58	.55	.55	.57	.62	.58	.58	.6		
6	.6	.6			.60	.64	.6	.7	.5	.50	.5	.55	.57	.55	.53	.55	.53	.6	.58	.53	.53	.55	.53	.54	.56	.55	.57	.55	.54	.55	.75		
8	.6	.6	.6	.71	.62	.60	.6	.6	.5	.53	.5	.55	.54	.55	.54	.50	.52	.53	.55	.53	.53	.55	.54	.54	.55	.54	.54	.53	.54	.6	.78		
10	.50	.6	.7	.7	.62	.64	.6	.6	.5	.55	.51	.54	.55	.55	.53	.55	.58	.58	.55	.55	.53	.56	.54	.56	.58	.55	.56	.54	.54	.60	.58		
12	.50	.6	.6	.6	.64	.64			.5	.55	.53	.54	.50	.58	.54	.55	.53	.62	.58	.54			.54	.55			.55	.54	.54	.58	.62		
2 AM			.6																														
4			.6																														
No.	10	10	9	10	10	10	9	8	8	10	10	10	10	10	10	10	10	10	10	10	9	9	10	10	9	9	9	10	10	10	10	300	No.
Ave.	.57	.62	.61	.67	.65	.65	.63	.61	.53	.57	.53	.53	.55	.55	.54	.54	.54	.53	.58	.56	.55	.56	.58	.58	.58	.55	.58	.58	.55	.58	.55	.58	Ave.
Max.	.6	.72	.7	.8	.69	.69	.74	.7	.6	.6	.58	.55	.7	.6	.58	.55	.58	.62	.64	.6	.59	.59	.73	.58	.64	.58	.58	.62	.58	.75	.78	.8	Max.
Max/Ave	1.05	1.17	1.14	1.19	1.06	1.05	1.17	1.14	1.13	1.05	1.07	1.03	1.25	1.09	1.05	1.02	1.08	1.11	1.13	1.10	1.08	1.05	1.30	1.05	1.45	1.05	1.05	1.10	1.07	1.31	1.34	1.45	

Turbidity Data Sept-Dec

Table B-4. Full Scale Operating Data, 2 MGD TRIDENT
Petrolia Ontario

Raw Water — November 1984

[illegible]

— Highest reading Nov 11"[illegible]"

Treated Water — November 1984

[illegible]

— [illegible] -Clarifier Flood Nov. 21

Turbidity Data Sept-Dec

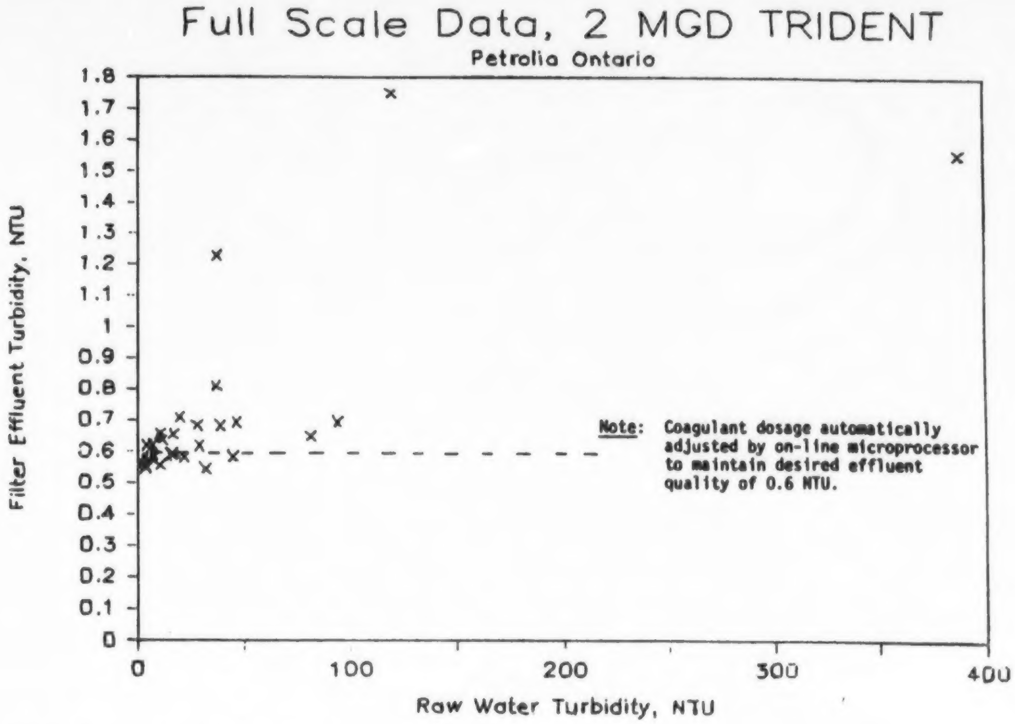
Table B-4. Full Scale Operating Data, 2 MGD TRIDENT
Petrolia Ontario

Raw Water — December 1984

[illegible]

Treated Water — December 1984

[illegible]



Note: Coagulant dosage automatically adjusted by on-line microprocessor to maintain desired effluent quality of 0.6 NTU.

Figure B-1. Full Scale Data Comparison of Raw Water Turbidity and Filter Effluent Turbidity at Petrolia, Ontario

APPENDIX C

Summary of Full Scale Plant Data From Lewisburg, West Virginia and Harrisburg, Pennsylvania

Table C-1. Full Scale Operating Data

Location: Lewisburg West Virginia
Plant: TRIDENT® TR-420A
Month: May 1985

	Turbidity, NTU				% Removal	
DATE	RAW	COAG	CLAR EFF	FILTER EFF	CLAR	FILTER
1	1.6	1.7	0.3	0.1	82.35	94.12
2	3.0	3.5	0.3	0.2	91.43	94.29
3	28.0	32.0	3.0	0.5	90.63	98.44
4	45.0	50.0	1.5	0.2	97.00	99.60
5	7.5	8.0	1.1	.2	86.25	97.50
6	6.5	7.0	.9	.2	87.14	97.14
7	4.0	4.5	.5	.1	88.89	97.78
8	3.0	3.8	.5	.1	86.84	97.37
9	3.0	3.0	.5	.1	83.33	96.67
10	2.0	2.5	.4	.1	84.00	96.00
12	1.8	1.9	.5	.1	73.68	94.74
13	4.0	2.0	.4	.1	80.00	95.00
14	45	50	1.1	.2	97.80	99.60
15	40	25	1.4	.2	94.40	99.20
17	5.5	6.	.6	.1	90.00	98.33
18	3.0	3	.6	.1	80.00	96.67
19	1.9	2	.8	.5	60.00	75.00
20	1.8	1.9	0.4	.2	78.95	89.47
21	1.5	1.3	.4	.1	69.23	92.31
24	40	37	6.0	.2	83.78	99.46
25	18	20	2.0	.1	90.00	99.50
26	10	12	1.1	.1	90.83	99.17
27	7.	7.5	.9	.1	88.00	98.67
28	4	4.5	.4	.1	91.11	97.78
29	3.5	4.0	.5	.1	87.50	97.50
30	3.0	3.5	.4	.1	88.57	97.14
31	3.0	2.0	1.1	.6	45.00	70.00

Table C-2. Full Scale Operating Data

Location: Lewisburg West Virginia
Plant: TRIDENT® TR-420A
Month: June 1985

	Turbidity, NTU				% Removal	
DATE	RAW	COAG	CLAR EFF	FILTER EFF	CLAR	FILTER
1	2.5	3.0	.6	0.1	80.0	96.67
2	2.0	2.5	.5	0.1	80.0	96.00
3	1.5	1.5	.3	0.1	80.0	93.33
4	1.2	1.4	.4	0.1	71.4	92.86
5	1.1	1.2	.6	0.1	50.0	91.67
6	1.0	1.1	.7	0.1	36.36	90.91
7	1.3	1.3	.6	0.5	53.85	61.54
10	5.0	4.5	.7	0.1	84.44	97.78
11	3.0	3.0	.7	0.2	76.67	93.33
12	5.0	5.0	1.1	0.3	78.00	94.00
13	4.0	4.0	1.0	0.2	75.00	95.00
14	1.3	1.4	.5	0.1	64.29	92.86
15	1.2	1.5	.5	0.2	66.67	86.67
16	1.3	1.4	.2	0.1	85.71	92.86
17	1.3	1.3	.4	0.1	69.23	92.31
18	1.7	1.5	.3	0.3	80.00	80.00
19	1.2	1.3	.3	0.2	76.92	84.62
20	1.5	1.4	.6	0.3	57.14	78.57
21	1.1	1.2	.4	0.1	66.67	91.67
22	0.8	.8	.2	0.1	75.00	87.50
23	0.7	0.7	0.1	0.1	85.71	85.71
24	.7	.8	.1	0.1	87.50	87.50
25	.7	.8	.2	0.2	75.00	75.00
26	.8	.7	.4	0.2	42.86	71.43
27	.7	.7	.3	.1	57.14	85.71

Table C-4. Full Scale Operating Data

Location: Harrisburg, Pennsylvania
Plant: Three 3MGD Clarifiers
Month: July 1985

	Turbidity, NTU					
DATE	RAW	CLAR EFF	FILTER EFF	CLAR % NET PRODUCTION	CLAR % TURBIDITY REMOVAL	% WASTE GENERATION
7/1	5.8	1.1	.14	95.91	81.03	4.43
7/2	5.9	0.9	.14	97.01	84.75	3.89
7/3	8.9	1.9	.15	97.09	78.65	3.80
7/4	10.4	2.4	.14	97.19	76.92	3.85
7/5	5.9	.8	.13	97.10	86.44	3.95
7/6	6.0	.9	.14	97.12	85.00	3.93
7/7	6.9	.8	.13	97.09	88.41	3.97
7/8	6.6	.8	.14	97.15	87.88	3.76
7/9	6.4	.7	.14	97.10	89.06	3.92
7/10	5.7	1.0	.17	97.16	82.46	3.88
7/11	5.8	1.0	.22	97.08	82.76	4.08
7/12	7.5	1.0	.19	97.08	86.67	4.00
7/13	6.8	.7	.15	97.14	89.71	4.52
7/14	6.8	.6	.14	97.10	91.18	3.78
7/15	7.6	.8	.17	96.79	89.47	4.14
7/16	10.7	1.0	.15	97.23	90.65	3.65
7/17	7.9	1.6	.16	97.21	79.75	3.78
7/18	9.6	2.00	.15	97.26	79.17	3.88
7/19	10.6	1.73	.25	97.18	83.68	3.70
7/20	8.7	1.0	.13	97.26	88.51	3.44
7/21	7.4	1.2	.16	97.28	83.78	3.77
7/22	15.1	3.4	.19	95.97	77.48	5.08
7/23	10.5	3.0	.32	95.77	71.43	5.64
7/24	7.7	1.6	.12	97.07	79.22	4.02
7/25	6.7	1.4	.14	96.98	79.10	4.01
7/26	9.6	1.4	.14	96.79	85.42	4.21
7/27	11.8	1.9	.14	96.08	83.90	5.43

BR/sg
9/20/85
Harris-Jul

Table C-3. Full Scale Operating Data

Location: Lewisburg West Virginia
Plant: TRIDENT® TR-420A
Month: December 1984

	Turbidity, NTU				% Removal	
DATE	RAW	COAG	CLAR EFF	FILTER EFF	CLAR	FILTER
1	6.0	12	2.5	.2	79.17	98.33
2	4.0	6.0	1.2	[illegible]	80.00	96.67
3	2.5	3.0	1.2	.2	60.00	93.33
4	1.5	2.0	.8	.2	60.00	90.00
5	1.5	2.0	.9	.1	55.00	95.00
6	1.7	2.0	.7	.1	65.00	95.00
7	1.4	1.8	.8	.1	55.56	94.44
8	1.5	1.8	.6	.2	66.67	88.89
9	1.5	1.9	.6	.1	68.42	94.74
11	1.8	6.0	2.5	.2	58.33	96.67
12	2.5	3.5	1.2	.1	65.71	97.14
13	2.5	2.5	1.2	.1	52.00	96.00
14	3.0	3.0	1.3	.1	56.67	96.67
15	2.0	6.0	.9	.1	85.00	98.33
16	2.0	2.5	1.1	.1	56.00	96.00
17	1.6	2.0	.6	.1	70.00	95.00
18	1.4	3.0	.9	.2	70.00	93.33
19	1.2	1.6	.4	.1	75.00	93.75
20	2.5	3.0	1.1	.2	63.33	93.33
21	11.5	12.5	2.5	.2	80.00	98.40
22	15	17.5	1.23	.2	93.14	98.86
23	16	18	1.8	.1	90.00	99.44
24	5.5	8.5	1.1	.1	87.06	98.82
25	20	18	2.5	.2	86.11	98.89
28	3	3.5	1.5	.3	57.14	91.43
29	3.5	3.5	1.1	.2	68.57	94.29
30	5	4.5	1.3	.2	71.11	9[illegible].56
31	2.5	4.0	.9	.1	77.50	97.50

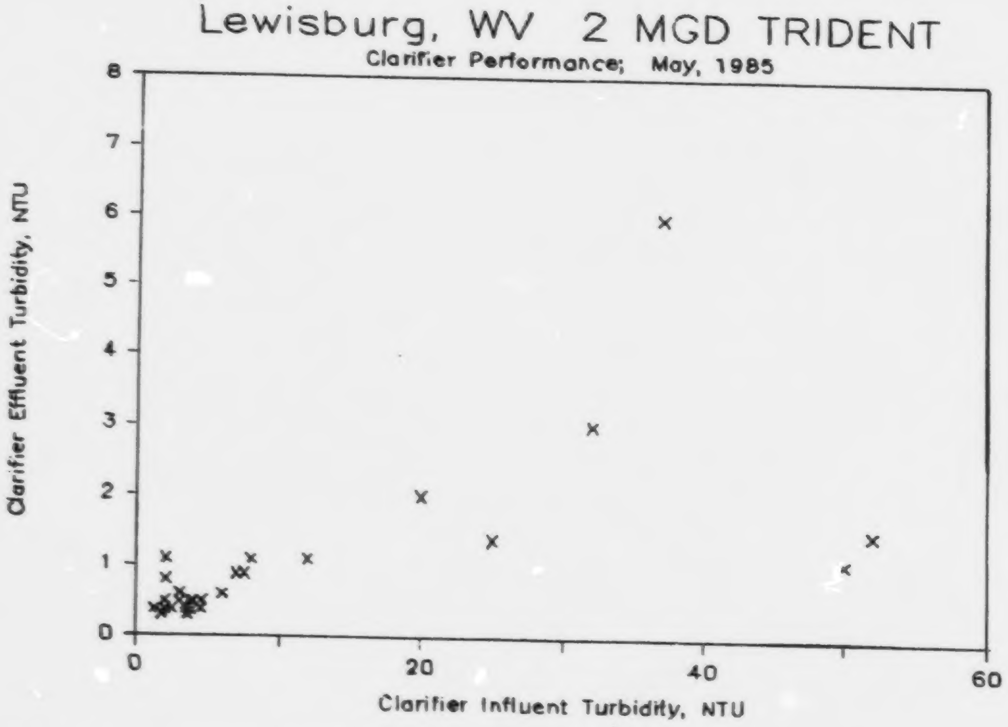


Figure C-1. Comparison of Turbidity for Clarifier Influent and Effluent

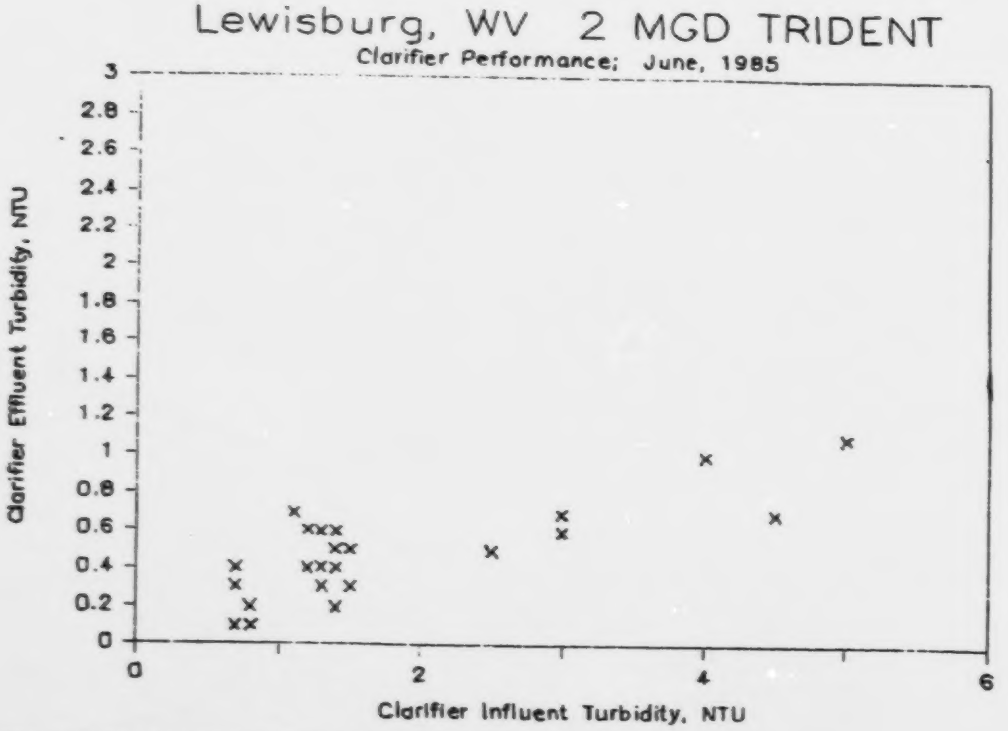


Figure C-2. Comparison of Turbidity for Clarifier Influent and Effluent

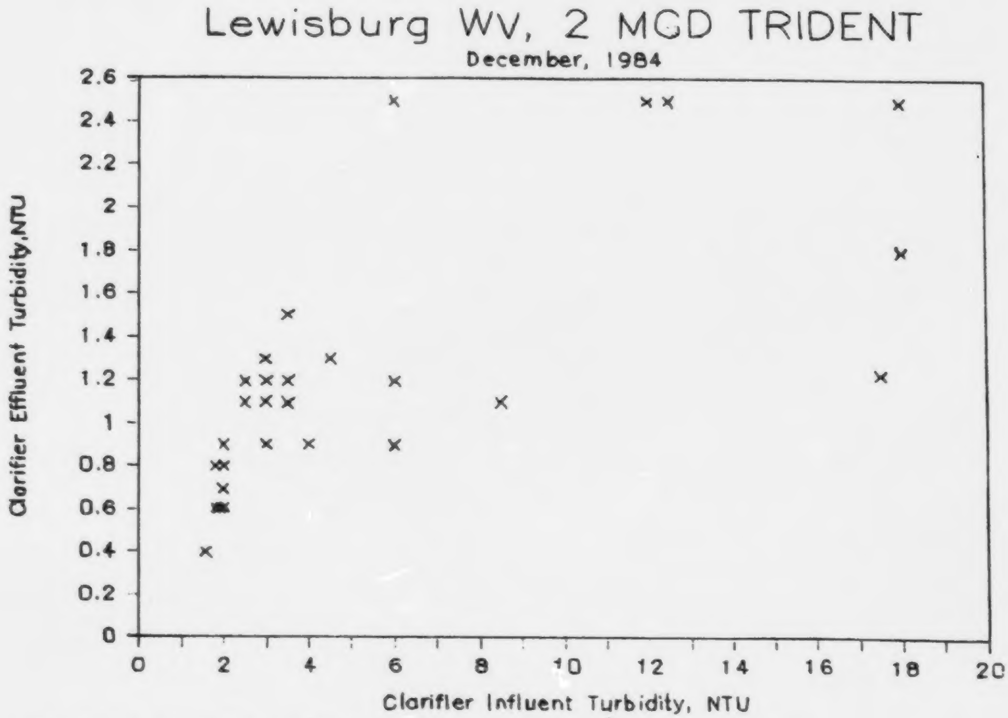


Figure C-3. Comparison of Turbidity for Clarifier Influent and Effluent

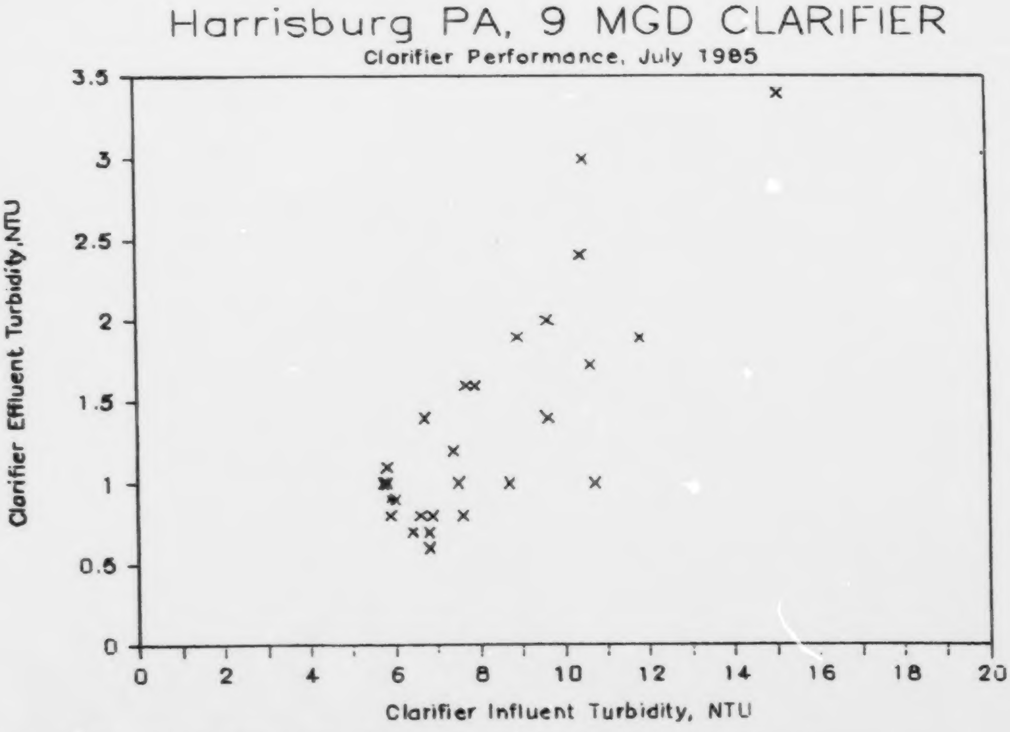


Figure C-4. Comparison of Turbidity for Clarifier Influent and Effluent

ABSTRACTS

Abstracts of papers not submitted for inclusion in the proceedings have been appended for reference purposes. Readers wishing to obtain copies of the complete papers are requested to make direct contact with the authors.

REFURBISHMENT OF EDMONTON'S ROSSDALE WATER TREATMENT PLANTS

by

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The City of Edmonton's main source of potable water is the treatment facility at Rossdale on the North Saskatchewan River. The Facility includes three partially separate plants with a total nominal capacity of 360 ML/d. The three plants were constructed between 1947 and 1965.

By 1980, the Facility was showing its age with a number of deficiencies being apparent. Piecemeal expansion of the 1947 plant to 1965 had also resulted in the City distribution system being served by three separate high lift pump stations of various ages and reliability. The Facility was also deficient in the amount of on-site storage of treated water.

This paper reviews the history of the Rossdale Facility from 1980 to 1986. During this period, studies, designs and construction took place under a comprehensive refurbishment program. The program included the following: extensive modifications to mixing systems, flocculation sections, clarifiers and filters, provision of new and refurbished chemical systems, expansion of control system, replacement of much of the existing underground, raw and treated water piping (including the installation of a new 1050 mm diameter ring main serving 7 feeders to the City distribution system), the construction of a 50 ML buried treated water storage reservoir and the integration of high lift pumping in a single new pump station containing six 1500 kw pumps, each having a design capacity of 90 ML/d. Of the six pumps, two are variable speed with speed control achieved by wound rotor motors and slip energy recovery systems. The station also contains 2-85 ML/d backwash pumps.

PLANNED MODERNIZATION OF WATER DISTRIBUTION SYSTEM USING COMPUTER CONTROL

by

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Municipal water distribution is a tremendously difficult and often thankless task. Control, reporting, historical data archive and retrieval, and continually rising generating costs are common problems. Many municipalities and operating groups are replacing manual and outdated telemetry systems with computer control systems to overcome these problems, but the acquisition of such a system has its own problems. All requirements are rarely met with any "off the shelf" product.

The common shortcoming of computer control system projects are over-runs, failure to achieve the performance expectations, and failure to reduce the operating costs in order to meet payback goals. Secondary problems include scheduling errors, interfacing incompatible equipment, nonacceptance by the user (distribution operators), and adverse owner publicity (the public).

These can be avoided.

Modernization using a distributed control system can be beneficial and virtually painless! The aim of this paper is to call on the author's experience in managing this type of improvement project from planning and system procurement stages through project execution and acceptance stages. These include:

Master planning
Precise specification
Vendor qualification
Vendor selection
Adequate training and documentation
User tools for future changes (after contract's over)
Spares and warranties
Continuous and near exhaustive testing
Realistic schedules

WATERMAIN & SERVICE FAILURES: SUGGESTED CAUSES & POSSIBLE PREVENTION

by

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T0A 3A0

Problems being experienced in the following areas will be dealt with.

1. Corroding service saddles off P.V.C. or A.C. mains.
2. Failures of service stop operating rods.
3. "Plug" blow outs and cracks of cast/ductile mains.
4. Corrosion of bolts on valves and fittings.

Our experience in controlling these above problems over the past three years in various municipalities in Alberta and over the past 12 years in Ontario will be presented and in particular the corrosion problems created when different metals e.g. brass, stainless steel, cast iron, steel, copper are connected together within an electrolyte (soil).

CORROSION PREVENTION OF MISCELLANEOUS AND STRUCTURAL STEEL IN WATER & WASTEWATER TREATMENT FACILITIES - ECONOMIC CONSIDERATIONS

by

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Corrosion occurs in spite of our best efforts. Appropriate design, plant operations and coatings of the steel are interdependent elements in the corrosion prevention program. Each element must receive appropriate consideration in order to ensure effective, efficient operation while maintaining appropriate aesthetics of the plant.

The coating element is perhaps the one that receives the least attention. Coating alternatives must be evaluated in detail at the design stage of the project. Detailed analysis must include an evaluation of:

1. Service conditions
2. Performance of various coatings in the respective service condition
3. Maintanance programs
4. Cost

Further, this detailed analysis must accommodate a great number of variables that are predicted to occur at sometime in the future and express them in terms of today's technology and costs.

REMOTE RE-CONNECTION OF SERVICE LATERALS & RELINING OF STORM & SANITARY SYSTEMS WITHOUT PAVEMENT DISTURBANCE

by

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Explanation of a method for reconnecting of service laterals to polyethylene relined mains by means of a patented method of reconnection using a grout method and cup drill, the work being performed from a small excavation in the boulevard area, or in some case from the clean-out in the house itself.

Also an explanation of the Buttress-Lok System which allows re-lining of mains utilizing a manhole to manhole installation with no excavation of any kind necessary, and is particularly suitable for, Highway Crossings, River Crossings, built up downtown areas and other areas where accessibility is a problem.

COST-EFFECTIVE MUNICIPAL WORKS

by

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This paper is based on the experiences of the author during over thirty years in consulting engineering in the municipal field in Western Canada. In keeping with the theme "Serving People and Saving Dollars", this paper is concerned with ways and means of designing and building municipal works effectively, but for less money, by following basic maxims such as: Always start at the beginning; don't do anything that you don't have to do, and KISS (Keep It Simple, Stupid). In an attempt to keep the talk interesting, the author has chosen examples from his experience to illustrate how engineers and others have planned and carried out municipal works that are cost effective.